GODFREY ON BUILDINGS

STEEL AND REINFORCED CONCRETE IN BUILDINGS

—BY—

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INTRODUCTION.

The purpose of this book is to supply a want in work where designing is done on a small scale that does not justify the employment of an engineer. A large amount of this sort of designing is done, and very much of it is faulty. While it may be to the interest of the author and his class to discourage designing on the part of men whose training does not fit them to do it more intelligently, the fact remains that the work is done and will be done, and done very often by men who do not understand much about the principles of proper design. The aim in writing this book is to lay down the principles of correct and consistent design as applied to buildings, and to give simple rules and tables to be used in designing.

Architects' designs for structural work of any magnitude should, of course, be checked by a structural engineer. The fee for this is less than for making an original design and may be included in the price of inspection. The checking of the details is another matter that can be best handled by a structural engineer: this can also be covered in a contract for the inspection of the steel work.

It is the author's intention, while indicating what may be safely done by one not thoroughly conversant with structural design, to indicate also, by the contents of the book, the line beyond which such a one ventures at his peril and to the jeopardy of life and property.

Bracing of buildings, while it is a matter of utmost importance, has been omitted from this book, for the reason that it is an engineering problem and one that can scarcely be standardized. In the majority of buildings bracing or stiffness is supplied by the walls. High or narrow buildings should be braced. The system of

bracing is a matter requiring special consideration, a matter for judgment and calculation and not for standards.

In the actual proportioning of a building generally the smaller details are designed first, that is, the floor system is decided upon first, then the floor beams are laid out, and their sizes as well as those of the girders are determined. Then the sections of the columns are worked out, and when the load on the base of a column is known, the pedestal and foundation may be proportioned. In this book the reverse order will be adopted in treating these parts, beginning with the foundation and going up and out toward the smaller details.

While this book is designed to be of special use to architects who have occasion to design in steel and reinforced concrete, it is believed that it will also be found useful to students and beginners as a preliminary to the author's more complete work on Steel Designing. There is also much in it that should be found convenient to structural designers in all lines.

An almost necessary accompaniment to this book is a book giving the dimensions and properties of steel sections, such as the Carnegie Pocket Companion or Godfrey's Tables.

CHAPTER I.

Foundations.

The area of a foundation in contact with the soil will depend upon the bearing power of the soil. This bearing power is best determined by experience rather than experiment, though in some cases experiments are resorted to. These are in the nature of a test load applied on a certain area for a given length of time. There are many features that must be taken into consideration in designing a foundation. The bearing power of a soil depends not only upon the nature of the soil itself, but also upon the degree of confinement of the soil. The degree of confinement will be gaged largely by the depth below the surface to which the trench or excavation is made. A clay that might stand safely two tons per square foot at six feet below the surface might heave and allow the same load to sink, if the trench is made only a foot deep. Moisture in a soil during construction has been the cause of disastrous settlement. Hence drainage at such a time is of prime importance. The basement floor of a building during construction is subject to repeated wetting, and may, if proper care is not taken, be the recipient of drainage from other ground. After completion of a structure the basement will be protected from moisture due to rains. If ground water is not naturally present, the soil will sustain much more load.

Another feature that should, if possible, be taken into consideration in planning a foundation is the possibility of excavation in close proximity to the foundation. If excavation is made near a foundation carrying a heavy load, and if that excavation extends to or below the level of the foundation in question, the soil may flow and allow large settlement of the structure. Thus, excavating for a neighboring building or a vault or subway may jeopardize the safety of a building that otherwise is quite safe.

Clay soils flow readily and are compressible. Sandy soils are not very compressible, but they will flow laterally, especially when wet, if not confined. Gravel is not compressible and is not so apt to flow. Mixtures of these in varying proportions combine the properties of each. Some clays, if kept perfectly dry, will bear heavy loads, but if wet, become like putty. Hence assurance that clay is dry or else confined is of great importance.

A good method of confining the soil under a structure to prevent flow is to drive sheet piling around it, thus holding the soil in a sort of box.

As far as practicable, where the soil is of a uniform carrying capacity, the pressure per square foot should be constant for the entire structure. Some settlement is to be expected, and it is important that this settlement be uniform over the entire foundation. When soils of different compressibilities are met with in the same building, such as clay and sand, the more compressible soil should have the larger footings.

The pressures allowed, by the New York Building Code, per square foot for various soils are as follows: Soft clay, one ton;, ordinary clay and sand together, in layers, wet and springy, two tons; loam, clay or fine sand, firm and dry, three tons; very firm, coarse sand, stiff gravel or hard clay, four tons. In Baker's Masonry Construction the following are given as the safe bearing power of soils in tons per square foot: Quicksand, alluvial soils, etc., 0.5 to 1; sand, clean dry, 2 to 4; sand, compact and well cemented, 4 to 6; gravel and coarse sand, well cemented, 8 to 10; clay, soft, 1 to 2; clay in thick beds, moderately dry, 2 to 4; clay in thick beds, always dry, 4 to 6; rock, from 5 up. This lower value is for rock equal to poor brick masonry. In case of hard rock the area of foundation may sometimes be determined by the strength of the foundation rather than that of the rock. Thus, if concrete is used in a pier with a bearing power of 15 tons per sq. ft., this sets the limit, though the rock may be capable of carrying a greater load.

Sometimes the compressibility of the soil is such that it is impracticable to give the footing the spread necessary for the load to be carried. Piles may then be driven and the load supported on these. Piles are sometimes driven to hard bottom and sometimes to a depth that results in a certain degree of refusal, depending in such cases upon friction of their sides for their supporting power. The usual loads allowed on wooden piles are 10 to 15 tons per pile. Sometimes as much as 20 tons is allowed on a pile. Piles supported by friction alone should not be loaded so heavily as those that are driven to hard bottom. Piles are generally kept 2½ to 3 feet apart as a minimum.

Wooden piles should be used only where they will be always wet, as they will rot if alternately wet and dry or if the soil is not constantly water soaked. In this case too, neighboring excavation should be anticipated if possible. Ground water level may be lowered by drainage subsequently made. Thus, in such locations as New Orleans, ground water level has been lowered by the construction of a sewer system.

Concrete piles, when properly made, are more reliable and durable than wooden piles and are capable of taking greater loads. Fifteen to twenty tons per square foot of sectional area may safely be allowed on concrete piles. The higher unit loads are for piles of larger diameter, as slender piles would act as columns to some extent.

The pressure on the footing for a wall is found by taking the load per running foot carried by that wall. This includes: the weight of the wall itself, making deductions for windows (say one-quarter or one-third of the area, depending on the circumstances;) the weight of the floors and roof bearing on the wall; the live or snow loads on floors and roofs supported on the wall. From this load per running foot of the wall and the allowed pressure per square foot the width of the footing is determined.

Footings under columns have the load of the column to carry and the load of the footing itself. The area is determined by the allowed pressure on the soil.

Concrete walls and footings are very much superior to rubble, because the monolithic character enables the former to settle uniformly. Settlement in a building is not of serious consequence, except when it is unequal settlement, and monolithic construction greatly reduces the possibilities of unequal settlement.

To effect uniform settlement, as stated, the unit pressure on the entire foundation should be made as near uniform as possible. Strictly, this cannot be done in ordinary cases because of the unknown and varying amount of the live load, also because of the fact that some of the walls or columns will have a greater or less proportion of their load as live load. Thus, the walls and exterior columns will have a more steady load because they take less of the floor load than the interior columns. One way to approximate equality of soil pressure is to make the areas of footings proportional to loads which include one-half or less of the total live load to be carried. This would necessitate somewhat greater area under the parts taking the smaller percentage of live load than the allowed soil pressure for its total load would demand.

When a structure rests on piles, uniformity of pressure is effected by spacing the piles to suit the intensity of the load carried. For example, if at one part of a wall the load carried is four tons per foot on piles that are good for 12 tons each, and in another part the load carried is three tons per foot, the spacing of piles should be three feet and four feet respectively. In large piers carrying unsymmetrical loads the spacing of the piles should be such that the center of gravity of the piles will coincide with the center of gravity of the load.

For a fuller discussion of foundation methods and designing the reader is referred to the author's book, *Concrete.*

CHAPTER II.

Footings.

The footings of walls and columns must of necessity have greater area than the walls and columns themselves. This spread must be effected in ways that will preserve the structural strength and distribute the load uniformly, or that will distribute the load so that the allowed pressure on the soil is not exceeded.

The simplest way to spread a wall footing is to increase the thickness of the wall by one or more steps at the base. In a brick or rubble wall the height of the step should be about four times the projection; or if the sides of the wall slope, the spread on either side should not be more than about one-quarter of the vertical height. The same relation should be observed in column footings of brick or rubble.

In a concrete wall or pier the projection or spread should be proportioned according to the allowed pressure on the soil by the following formula:

$$sp^2 = h^2 \qquad (1)$$

where s is the pressure in tons per sq. ft. allowed on the soil, p is the projection of the wall or pier and h is the height in which the step or slope p occurs.

For derivation of these relations, as well as those that follow, bearing on reinforced concrete footings, see the author's book *Concrete*.

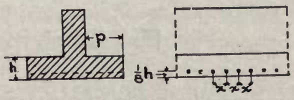


Fig. I.

A wall footing may be made of reinforced concrete as shown in Fig. 1, with the following relations:

$$h = .35\, p\, s \tag{2}$$

$$p = 50\, d \tag{3}$$

$$x = 9\, d \tag{4}$$

where s is the allowed pressure on the soil in tons per sq. ft., and d is the diameter in inches of square reinforcing rods. The projection p will be found from the load per running foot on the wall and the allowed soil pressure. Then from equations (2), (3), and (4) the other dimensions may be found. Assuming $p=4$ ft. and $s=1\frac{1}{2}$ tons, h will be 2 ft. $1\frac{1}{2}$ in. The reinforcing rods would be one inch square, spaced 9 in. apart.

This sort of footing is appropriate chiefly where the soil is of low bearing power, since the height h required for shear where heavy pressures are considered will usually make reinforcement uneconomical, as a somewhat greater height will make reinforcement unnecessary.

Any footings in reinforced concrete must be made of sloppy concrete, as no other will grip and protect the steel. Dry or rammed concrete is quite unsuitable for reinforced work.

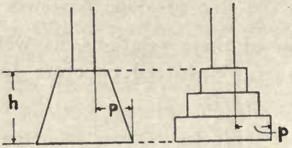


Fig. 2.

Column footings may be made in plain concrete as shown in Fig. 2 with either stepped or sloping sides. The relation between p and h may be the same as given in Equation (1).

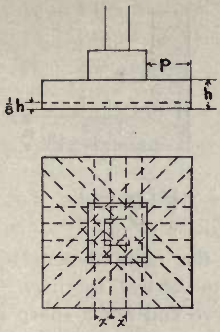


Fig. 3.

A reinforced concrete footing should be made as shown in Fig. 3. All rods should pass under the upper plinth. There are designs in which the rods are spaced equally out to the edges of the rectangle. This is poor design, as the rods near the outer edges can do little or nothing.

In this footing, with s as before:

$$h = .5\, p\, s \tag{5}$$

Equations (3) and (4) apply as in the wall footing.

When the outside line of a wall is the property line, of course all offsets must be made on the inside. If these offsets are not large, the pressure on the soil may be considered as uniformly distributed. When the wall is not a long one, or where there are cross walls, a projection of considerable width could be made without the necessity of assuming eccentric load on the foundation.

If the projection of a wall is wide as in the L-shaped wall shown in Fig. 5, unequal pressure on the soil must be considered. The resultant pressure must fall within the middle third of the base. The size and spacing of rods for this projection, as well as the width and depth of the pro-

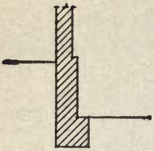


Fig. 4.

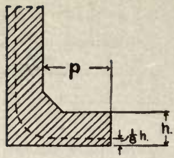


Fig. 5.

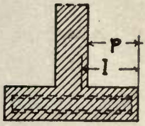


Fig. 6.

jection, may be of the same dimensions as those given under Fig. 1 for symmetrical footings. The rods should be given an easy curve and not a sharp bend. The radius of the curve should be about 20 times the diameter of the rod. Rods should run up into the wall as indicated for anchorage. Anchorage for a rod requires embedment in concrete for a distance equal to 50 times the diameter of the rod.

The projection p in the wall, shown in Fig. 5, must be less than twice the thickness of the wall, that is, the resultant pressure must come under the wall itself, so as to prevent, or at least minimize, bending in the wall itself.

Wall footings are sometimes made by using steel beams or rails as needle beams, as indicated in Fig. 6. Rails are not economical for this purpose, because they are much heavier for the same strength than I-beams.

The size of I-beams necessary for any given case is found as follows:

The upward pressure on the soil is considered as a uniform load on the beam. The beam is a cantilever with an overhang or a span l. This distance l is a few inches more than the projection p, say 2 to 6 in., depending on the magnitude of the footing. The load that a beam can sustain as a cantilever of a span l is just one-quarter as much as that which it can sustain as a simple beam of the same span. Turning to Chapter VI, Table II, it is seen that the capacity of an I-beam of any span is found by dividing the quantity Q in the table by the length of that span in

feet. This capacity is in tons of total load carried by the beam as a simple span. It must be divided by four to find the safe load that the beam can take as a cantilever. If the operation be reversed, we would multiply the load on the cantilever by four and then by l to find the value of Q. For example, if l is four feet and the upward pressure of the soil is two tons per sq. ft., we find Q to be 4x2x4x4=128. This is the value per running foot of the wall. As Q, for a 10″ I 25 lb. is 130, we could use a 10″ beam every foot.

These needle beams must be completely surrounded with concrete.

Grillages for column footings are often made as shown in Fig. 7. In this grillage the load of the soil is first taken by the lower tier of beams to the upper tier; it is then delivered by the upper tier to the column base. The span of the lower tier is the distance from the center of outer beam of the upper tier to the edge of the footing. The span of the upper tier is the distance from the edge of the footing to a point a few inches within the column base, as indicated.

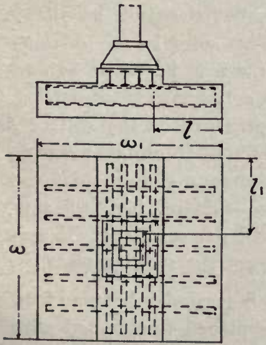


Fig. 7.

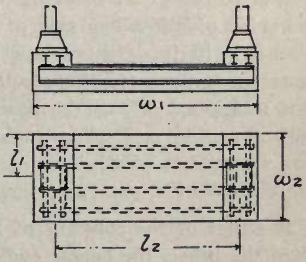


Fig. 8.

As an example, suppose it is desired to proportion the beams for a column footing in which w is 10 ft., w_1 is 8 ft., l is 3 ft., and l_1 is 4 ft., the upward pressure of the soil being 4 tons per sq. ft. The load taken by the lower tier of beams as a cantilever of span l is 10x3x4=120 tons. Multiplying this by 4 and by the span l we have for the aggregate value of Q for the set of beams 1,440. We could use 5-15″ 42-lb. beams, for which Q is 1,571. For the upper tier of beams the load carried is w_1xl_1, as the lower beams deliver this area of load into the upper beams. Q for this set of beams is then 8x4x4x4x4=2048. We could use 4-20″ 65-lb. beams, for which Q is 2,263.6.

These beams would have separators with bolts running through the set. Each would have about three lines of these separators.

Very often in wall columns only one set of beams will be used under the column base. The size of these will be found in the same way as for the grillage beams.

Sometimes, on account of keeping the column footing within property lines, two columns are built on the same grillage as indicated in Fig. 8. Here the four beams of the upper tier take the cantilever load on the area w_1xl_1, and are designed as before. The lower beams carry the upward pressure on the area w_2xl_2, but they act as simple beams and not as cantilevers. The span is the distance center to center of columns, for there is but little balancing load on the other side of the columns. If w_2 is 6 ft. and l_2 is 18 ft., with an upward pressure of the soil of 3 tons per sq. ft., Q=18x6x3x18=5,832. (Note that we do not multiply by 4, as the beams act as a simple span and not a cantilever.) We could use 6-24-in. 80-lb. beams, for which Q is 5,568. This is about 5 per cent. shy. Beams weighing 90 lbs. per foot, would meet the requirements.

If either of the columns of Fig. 8 carried a heavier load than the other, the beams could be placed fan-shaped with the center of gravity of the footing corresponding with that of the combined load.

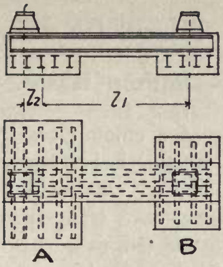


Fig. 9.

Another way to take care of the footing of a wall column is illustrated in Fig. 9. Here a lower tier of beams is provided for each column, but the upper beams have the added office to perform of carrying the load of the wall column back to the middle of its grillage. This load is carried on the beams as a cantilever with an overhang *l* 2. The load is the concentrated load of the column. A beam acting as a cantilever of a given span supporting a load at its outer end will sustain only one-eighth as much total load as the same beam acting as a simple span with the load uniformly distributed. Hence, to find *Q*, we would multiply the load by the span and by 8. For example, suppose *l* 2=4 ft. and the column load is 70 tons. *Q*=70x4x-8=2,240. This would require 3-20 in. 80-lb. beams, for which Q is 2,347.2.

Where the depth permits of a deep girder being used, a plate girder or a box girder is more economical than beams for heavy column loads. A column may be riveted between the webs of a box girder, which acts as a cantilever to carry the load to a grillage, located within the property line.

CHAPTER III.

Column Bases.

Usually the foot of a column rests on a separate cast base. The reason for this is because the cast base can be set up on the foundation and leveled and brought to a proper elevation much more easily than a column. It can also be more readily located, as a mark can be made at the center of the base on the planed top of the same.

The cast base is invariably planed on top. Sometimes it is also planed on the bottom; but more commonly the bottom is left as cast, and the base is set in cement mortar or is shimmed up to its proper level and grouted through holes in the bottom.

In the design of a cast base the first consideration is to have area enough in contact with the masonry so that the pressure on the same will not be excessive. A pressure of 300 lbs. per sq. in. may be allowed on concrete. This will give a basis for finding the area of the base. Thus, a load on the column of 150,000 lbs. would require a base of 500 sq. ins. A round cast iron column could have a base 26 ins. in diameter, or a steel column could have a square base 23 ins. in diameter.

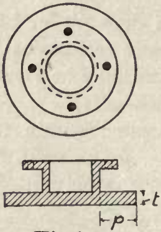


Fig. 1.

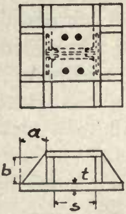


Fig. 2.

The usual design of a base for a cast iron column has an upper flange to which the column is bolted and a lower plate resting on the masonry. This plate, as in all other masonry bearing plates, should have no unstiffened projection greater than about twice the thickness of metal in cast iron or four times the thickness in steel. That is, *p* in Fig. 1, should not exceed 2 *t*.

When there are stiffening ribs, as in Fig. 2, the spacing of ribs or the thickness of the base plate should be governed by the relation of *s* to *t*. In cast iron *s* should not exceed about four times *t*, and in steel *s* should not exceed about eight times *t*.

The relation of *a* to *b*, to give the proper slope to the rib, depends upon the thickness of rib and base plate. If *a* be made equal to *b* in a cast-iron base, the stresses will generally not be excessive. In a cast-steel base *a* may be about 1.5 times as great as *b* without giving excessive stresses. Usually, however, the value of *a* is made relatively less than these ratios would show.

Another feature of a cast base that should receive attention is the location and shape of the vertical webs under the shaft of the column. If the column is of an I shape, these webs should be approximately the esame shape, as shown in Fig. 2. A column approximately square in shape should have the webs of the base formed in a square box. It is a good plan to have a good sized hole in the bottom of this box for grouting and a number of other holes for the escape of air.

In large bases holes are usually left for grouting. If, as intimated in the last paragraph, a vertical opening be left at the middle of the base, this can be filled with grout to act as a sink head to give pressure to the grout. If the grout be allowed to rise in other openings in the base, a better filling of the space is assured than if grout is poured in several holes at once. The latter method allows entrapped air to form pockets under the base.

Column bases in buildings are usually laid on the concrete footing without being anchored or bolted thereto.

CHAPTER IV.

Columns and Other Compression Members.

Building columns may be of wood, cast iron, steel or reinforced concrete. After the following discussion on the method of finding the load carried by a column, the methods of designing the columns of these several classes will be taken up.

The load taken by a column at any given floor or roof level would of course be the sum of the loads delivered to it by the beams, girders or trusses connecting to the column at that level. But to find the reactions of all of these would generally be very tedious work. The usual method is to find the area of floor and the length of wall tributary to the column and from suitable units for dead and live load to calculate the load delivered at each floor level.

The load per square foot of the floor construction must include floor covering, sleepers, filling, arches or slabs, an allowance for beams, and an allowance for girders. The allowance for beams is a load per sq. ft. that will cover the weight of the beams. Thus, if 25-pound beams are spaced 5 ft. apart, this allowance is 5 lbs. If girders, weighing 45 lbs. per ft., are spaced 15 ft. apart, 3 lbs. would be allowed for the girders. The area tributary to a column is the surface of floor that the column carries. It is usually a rectangle bounded by lines midway between this column and the next in each of the four directions (or midway between the column and the wall).

The area for live or superimposed load is the same as for dead load. Ordinary partitions are usually considered as covered by the live load allowance. However, it is well to make an allowance, of say 5 lbs. per sq. ft. in the dead load, to cover the weight of partitions. Extra heavy partitions should be estimated. Any interior brick walls

should be allowed for by finding the reactions of the beams supporting the same.

When the walls are carried by the columns, the full weight of wall may be estimated and the windows deducted; or, if the window openings are fairly regular, an estimate may be made of the proportion of solid wall, the load on the column being calculated from this. Of course each column will carry a length of wall equal to half the sum of the distances to the next adjacent columns, and a height equal to that to the next wall beam above or to top of wall. Ordinary brick walls weigh about 10 lbs. per sq. ft. for each inch in thickness. Stone walls weigh about 12 or 13 lbs. per sq. ft. for each inch in thickness.

Where possible, columns should be symmetrically loaded, as unsymmetrical loads produce bending moments in the column, and these are seldom provided for in proportioning the section of the column. In interior columns balance of the loads is usually easily accomplished. In wall columns a practical balance can be effected by attaching the wall beams to the outer side of the column and the floor beams or girders to the inner side. The most economical and satisfactory method of offsetting the effect of a heavy eccentric load on a column is to make a deep riveted connection of the girder to the column. This puts the bending stress into the girder that would otherwise have to be taken by the column, and the girder is generally amply able to carry the bending stress. The riveted connection may be for the full depth of the girder, or it may be made greater than the depth by use of gusset plates or corner brackets. In a rolled beam top and bottom riveted flange connections aid greatly in overcoming bending due to eccentric loads.

Wooden Columns. The allowed load in direct compression on a wooden column is very simply found. It depends upon the ratio of the free height of the column to the least width. This ratio of free or unsupported height to width must be clearly understood, however. In a simple post without braces from base to top the free height is the full

length of the post. In posts having knee braces or struts connecting to some part of the building capable of offering ample resistance, the free height is the distance from the base to the point where the braces connect.

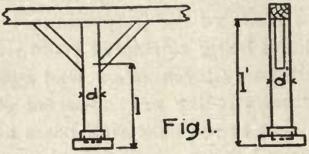


Fig. 1.

If the braces hold the column in only one direction, as in Fig. 1, there will be two ratios to consider, namely: l/d and l'/d'. The smaller of these two ratios will be the governing factor in determining the strength of the column. It is to be observed that the braces must be capable of holding the column in line. Two equally strong or equally weak columns braced together by a horizontal brace would not be shortened in their effective length by such a brace.

When the ratio of length to width of a wooden column is known the allowed load per square inch is as follows:

For yellow pine or oak ....1,000—18 l/d
For white pine .... 800—15 l/d

In the following table the allowed load per sq. in. is shown for three different ratios.

TABLE I.

STRESSES PER SQ. IN. ALLOWED ON WOODEN POSTS.

Ratio	Yellow Pine or White Oak	White Pine
30	460	350
20	640	500
10	820	650

For example, suppose an 8x8 yellow pine post is 10 feet long. The length is 15 times the width and the unit compression allowed is 730 lbs. per sq. in. This post would carry safely 730x64=46,720 lbs. A white pine post 6x8 in section and eight feet long would have a ratio of length to least width of 16. A load of 560 lbs. per sq. in. could be allowed, or a total load of 26,880 lbs.

Generally, wooden posts should not be less in width than 1-30 of the length.

The base of a wooden post or column is sometimes made of cast iron. A socket is cast in the base into which the post fits. The spread of this cast base must be such as to keep the pressure on the masonry within the allowed limits. Thus, the 8x8 post of the last paragraph with its load of 46,720 lbs., if 250 lbs. per sq. in. be allowed on the masonry, would require 187 sq. in. of base. A base 14 ins. sq. would do for this column. As the projection around the column is 3 ins. the thickness should be half of this or 1½ in.

Cast-iron caps are very often used at the tops of columns to act as splices and as seats for girders. Steel plates or angles would be very much better, as cast iron is brittle and liable to be broken by the concentration of the beam load on the edge of the bracket. Fig. 2 shows a suggested detail.

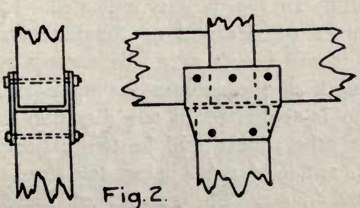


Fig. 2.

Cast-Iron Columns. The allowed load in direct compression on a cast-iron column is found in a similar manner to that on a wooden column. It is true that there are many formulas for the strength of a cast-iron column, but they are for the most part highly theoretical and their allowed unit loads are not borne out by tests. A few simple rules for designing and a simple formula for the allowed compression are all that a material such as cast iron demands.

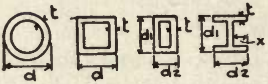


Fig. 3.

Fig. 3 shows the common sections used in cast-iron columns. The round and square shapes are generally used for interior or exposed columns. The oblong column may be used in a wall or between windows. The H-shaped column may also be used between windows.

The thickness t should ordinarily be not less than about ½ in. In the H-shaped column the thickness t should not be less than about one-fifth of x.

The allowed unit stress on cast-iron columns should not exceed

$$7{,}600-40\ l/d$$

where l is the unsupported length of the column and d is the least width. In Fig. 3, d is indicated. It will be the outside diameter of a round or square column. In the other shapes it will be d_1 or d_2, depending upon the unsupported length of the column for these two directions. If the column is supported in one direction and not supported in the other, there will be two ratios to consider, namely: l_1/d_1 and l_2/d_2; l_1 being the free length corresponding to d_1, etc. The smaller of these two ratios will determine the unit load to use on the column.

From the foregoing unit stress the allowed load per sq. in. on cast-iron columns may be found for various ratios and tabulated as follows:

TABLE II.

STRESSES PER SQ. IN. ALLOWED ON CAST-IRON POSTS.

Ratio	Allowed Stress	Ratio	Allowed Stress
40	6000	20	6800
30	6400	10	7200

Generally, cast-iron columns should not be less in width than 1-40 of the length.

For convenience in finding the areas of hollow square and circular columns, the following table is given. The column area will of course be the difference between the inner and the outer circle or square. When the outside diameter and the desired area are known, the area of the inner circle or square will be the difference between that of the outer circle or square and the required area. From this the inner diameter can be found in the table.

TABLE III.

Areas of Squares and Circles.

Dia.	Area Ro'nd	Area Sq'are	Dia.	Area Round	Area Square	Dia.	Area Round	Area Square
3	7.069	9.000	7	38.485	49.000	11	95.033	121.000
3¼	8.296	10.563	7¼	41.283	52.563	11¼	99.402	126.563
3½	9.621	12.250	7½	44.179	56.250	11½	103.869	132.250
3¾	11.045	14.063	7¾	47.173	60.063	11¾	108.434	138.063
4	12.566	16.000	8	50.266	64.000	12	113.097	144.000
4¼	14.186	18.063	8¼	53.456	68.063	12¼	117.859	150.063
4½	15.904	20.250	8½	56.745	72.250	12½	122.718	156.250
4¾	17.721	22.563	8¾	60.132	76.563	12¾	127.676	162.563
5	19.635	25.000	9	63.617	81.000	13	132.732	169.000
5¼	21.648	27.563	9¼	67.201	85.563	13¼	137.886	175.563
5½	23.758	30.250	9½	70.882	90.250	13½	143.139	182.250
5¾	25.967	33.063	9¾	74.662	95.063	13¾	148.489	189.063
6	28.274	36.000	10	78.540	100.000	14	153.938	196.000
6¼	30.680	39.063	10¼	82.516	105.063	14¼	159.485	203.063
6½	33.183	42.250	10½	86.590	110.250	14½	165.130	210.250
6¾	35.785	45.563	10¾	90.763	115.563	14¾	170.873	217.563

Cast-iron columns are not to be recommended for buildings of more than about three or four stories in height. They should not be used in any case in a building whose lateral stability depends in any wise on the columns, such as one whose exterior walls are carried by the metal frame. Cast iron lacks toughness and should be used only in simple compression in columns and in situations where there is little or no bending stress.

Given an example where the wall between two buildings is to be removed and replaced by cast-iron columns. Assume the width of each building to be 20 feet; the height of the first story 14 ft.; three stories above this of 11 ft. each; thickness of wall 13 in.; total weight for floors 150 lbs. per sq. ft.; total weight for roof 120 lbs. per sq. ft.; spacing of columns 18 ft.

Each column will carry the following load:

18 ft. of wall, 33 ft. high............=18x33x130=	77,220
20x18 ft. of roof, at 120..............=20x18x120=	43,200
3 floors, 360 sq. ft. each, at 150......=3x360x150=	162,000
	282,420

Assume a round section of column 12 ins. in outside diameter. The ratio l/r is 14/1 or 14. The allowed load per sq. in. is 7,040 lbs. The area required is 40 sq. ins. A circle 12 ins. in diameter has an area of 113 sq. ins. This leaves 73 sq. ins. as the area of the inner circle, or say a 9.5-in. circle. This gives a thickness of metal of 1¼ in.

At the top of this column there will, of course, be pairs of I beams or a box girder to carry the load of the wall and the floors above. These beams would not have to be designed to carry all of this load as uniformly distributed, because the rigidity of the solid wall would allow much of it to be carried by the wall directly to the columns. Of the 141 tons on a pair of beams of a span of 18 ft., we may assume 100 tons as a uniform load on a pair of beams. The value of Q in the table of the capacity of beams is

then 100x18=1,800. Two 24-in. 80-lb. beams would be used. These have a combined value of Q equal to 1,856.

The base of this column should not be made to rest directly on the foundation wall, but should have distributing beams so that the pressure on the wall will not be excessive. If two beams be used, each 10 ft. in length, the load per foot on the pair of beams will be 141÷10 or 14.1 tons per ft. The beams will have a cantilever span of about 4.5 ft. Each I beam will have a load on this cantilever of 7.05x4.5=31.7 tons. For the value Q of the table this is to be multiplied by 4 and by the span 4.5, or Q=31.7x4x4.5=571. There will then be required 2-15″ 80-lb. beams.

The area of the flanges of these beams in bearing on the wall is 2x6.4x120=1,536 sq. ins. This is a pressure on the wall of 282,420÷1,536=184 lbs. per sq. in. The wall should have a concrete or a cement mortar finish in which to bed the beams.

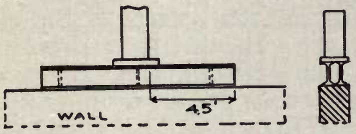


Fig. 4.


Fig. 5.

Splices. Cast-iron columns are generally spliced by four or more bolts through flanges. The flanges are made of about the same thickness as the shell of the column. The splice is made about at the floor level. The flanges should be about 2½ or 3 inches wide to allow space for bolt heads. The bolt holes should be drilled and not cored. The ends of columns should, of course, be planed true.

Where a change in section of cast-iron columns occurs, provision must be made for carrying the load from the upper to the lower section. This may be done, as in Fig. 5, by making extra heavy flanges, stiffened with ribs, on the upper column.

Generally, the shaft of a cast-iron colunm, should be uniform from end to end of the column. If the column is flared out for an ornamental head or base, it should be strengthened by inside ribs to carry the column load.

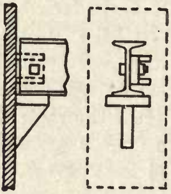


Fig.6.

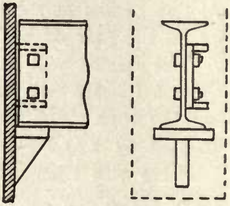


Fig.7

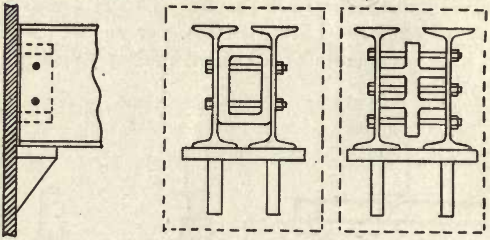


Fig.8

Beams generally connect to cast-iron columns by means of brackets on which they rest and lugs for bolted connection to the web. The brackets are usually made as indicated in Figs. 6, 7, 8. These brackets should project about 3 or 4 ins. from the face of the column. There is no advantage in a wide shelf, but rather the reverse, as the beam is apt to bear on the outer edge and produce heavy bending stresses on the bracket. There should be a stiffening rib under each beam, not less than twice as deep as the width of the bracket. The shelves and ribs should have a thickness of metal about equal to that of the shell of the column, but not less, for ordinary work, than about one inch. The shelves are not planed, but are cast smooth; the bolt holes are usually cored.

Eccentric or unbalanced loads should be guarded against in cast-iron columns, because of the lack of toughness in the metal.

Steel Columns. There are many forms of steel columns from the single angle up to the built column of several hundred square inches of sectional area. The selection of an appropriate style of column for any given case will depend upon the several conditions of the case.

There are many column formulas that purport to give the correct load that will cause ultimate failure in a column or the correct safe load; but, excepting the formulas of the form known as the Euler formula, these usually bring in empirical "constants" that are, in fact, not constant and that depend upon conditions that cannot be made uniform in commercial work.

A steel column acts partly as a spring to resist bowing and partly as a shaft in compression to resist crushing. The ultimate strength of a slender column can be calculated closely, but the ultimate strength of a shorter column can only be very roughly approximated. The ratio of slenderness of a column is the ratio between the length and the least radius of gyration of the cross section. The large majority of compression members have ratios of slenderness varying between 30 and 150, and it is between these limits that the greatest uncertainty as to calculated strength exists. When a compression member is very short, its ultimate unit strength is nearly equal to the ultimate unit strength of cubical specimens; when the member has a ratio of slenderness of 150 or more, its ultimate strength is the definite value shown by the Euler formula.

A few words are deemed advisable here in the way of warning to the inexperienced designer. It is often asked, "What is the factor of safety of a certain structure?" and the answer usually given is 4 or 5, according as the designer thinks he has split up the ultimate strength of his members into 4 or 5 parts. The builder may say that he is satisfied with a factor of safety of 3 or less, and the designer is asked to cut down his sections accordingly. This

is a dangerous undertaking, especially when the commonly used column formulas are taken at their face value. As the author has shown in Railway Age-Gazette, July 2, 1909, the Gordon-Rankine column formula shows apparent ultimate strengths of columns that are in some cases more than 100 per cent too great. This subject is more fully treated in the author's Structural Engineering, Book III.

In some manufacturers' handbooks the supposed ultimate strength of columns is worked out on the basis of the Gordon-Rankine formula for values of the ratio

$$\frac{\text{Length in feet}}{\text{Radius of gyration in inches}}$$

as high as 20 or more. This is an actual ratio of slenderness of 240. It is entirely too slender for a practical column. Furthermore, the ultimate strength given for a pin-ended column of this ratio, is nearly 12,000 lbs. per sq. in. The actual ultimate strength of this column is 5,000 lbs. per sq. in., even if the column be made of the highest grade and hardest steel that it is possible to manufacture.

Designers are warned against using columns or other compression members of a ratio of slenderness greater than about 150. Some specifications and building codes do not allow a greater ratio than 120.

What is known as the straight-line formula for the strength of a column is better than formulas of the Gordon-Rankine type, because the straight-line formula shows very low strength for slender columns and because it agrees more nearly with tests.

A straight-line formula in common use for building work gives a unit stress per sq. in. equal to

$$15{,}200—58\ l/r.$$

where l is the length in inches and r is the least radius of gyration in inches. In a well-built and properly designed and centrally-loaded column, this formula gives the load that can safely be sustained. The factor of safety is a matter depending entirely on the perfection of the work

and is a value quite impossible to determine. Designers are cautioned to adhere to the formula.

The length l in the column formula is, of course, the unsupported or unbraced length of the column or other compression member. As explained heretofore in this chapter, there may be two or more ratios of slenderness to consider. A compression member may be braced in one direction and free to buckle or bow in another direction. Steel compression members may be of unsymmetrical sections, as in the case of a single angle or zee bar; in such case the diagonal radius of gyration must be found, as this is less than the radii on the rectangular axes. A single angle or zee bar would fail by bowing in a diagonal direction.

Single channels and single I-beams do not make good compression members, because the radius of gyration with the neutral axis parallel with the web is so small. In general, these should not be used as compression members, unless they are braced at close intervals, or bolted to a wall, or built into a wall.

Tables IV to XX give the total load allowed on compression members of various shapes. These tables should be used with caution and a knowledge of their limitations. Correct design and proper end details of columns are essential to produce a column that will have safe carrying capacities as shown in the tables.

The heavy zig-zag lines in the several tables show the limits of safe length of columns at about 120 times the radius of gyration. Preferably the length of column should be kept within this limit. In some cases the ratio may be made as high as 150, when values to the right of the zig-zag line apply. The value of the radius of gyration of nearly all of these sections may be found in Godfrey's Tables. The following rules apply approximately for some of the sections:

For the star-shaped sections shown in Table IX, the value of r is about four-tenths of the width of the leg of

one angle. The limit of 120 times r is then about 48 times the width of the leg of one angle. Thus, for 4—4″×4″ angles this limiting length would be 16 ft.

For gas pipe the radius of gyration is about .35 of the outside diameter. At 120 radii the unsupported length is then about 40 times the outside diameter.

For Bethlehem H Sections the radius of gyration is about .4 of the width B of flange, hence 120 radii is about 48 times the flange width.

For the sections shown in Tables XII and XIII r is about .20 to .22 times the width of flange, hence 120 radii is about 25 times the flange width.

For the channel columns of Table XVI r is about .4 of the depth of channel, hence 120 radii is about 48 times the depth of channel.

For the zee-bar columns r (minimum value) is about .62 times the web of one zee bar. The limit of column length, at 120 radii, is 18.5 ft. for 3-in. zees, 24.5 ft. for 4-in. zees, 31 ft. for 5-in. zees, and 37 ft. for 6-in. zees.

Tables IV and V give the strength of single angles in compression, but in order to develop the strength shown in these tables the angles should preferably be milled on the ends. They need a square end bearing, so that the load will not be eccentric. Connection by means of rivets through each leg of the angle may be sufficient to balance the load, but that connection should be to rigidly held parts. Single angles should generally be avoided as members of a truss, but if used, the allowed stress should be only about half of that shown in the table, so as to allow for eccentricity. This is true, whether or not both legs of the angle are connected with rivets at the ends. When the stress is applied to the end of an angle by a gusset plate, extra lug angles connecting to the outstanding flange do not centralize the stress from the gusset plate.

When a single angle used as a post has a channel riveted to each flange, as in Fig. 10, a good rigid end connection is obtained, and, if the base of the post is milled and has a square bearing, the post may be taken as good for the

value in the table. If the angle is not milled on the end, its value in compression may be determined by the rivets in the lugs at the end. Very frequently these angles are simply sheared off at the ends and do not bear against the base plate.

Figs. 9, 10 and 11 illustrate small angle posts. When two angles are used as a post or compression member, they should be riveted together at intervals of a foot or two, so that they will act together as one member. When they are separated, as in a truss by the thickness of a gusset plate, washers are used between the angles at these rivets.


Fig. 9.

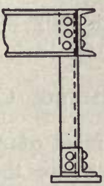


Fig. 10.

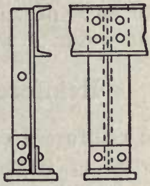


Fig. 11.

Compression members are sometimes made of two angles separated several inches and joined by small batten plates at intervals of three or four feet. These do not make good compression members, unless they are considered as two separate angles, and the ratio of slenderness so taken. When the two angles are joined by lattice from end to end of member, they may be considered as one member, for then the triangular system of lattice bars compels one angle to aid the other in resisting buckling from end to end of the member.

Gas pipe posts usually have a threaded cast iron flange for an end connection.

Beam connections to steel columns and column splices will be considered in another chapter.

In selecting the sections for columns in the successive stories of a building, they should be arranged so that the metal of the upper section will bear against metal of the lower section, unless special provision is made in the splice. Frequent changes in the general outside dimensions of the columns should not be made, as these involve special splices and more irregular beam connections. In closed channel sections the thickness of cover plates and the weights of channels may be reduced, using the same depth of channel for several tiers. In I-shaped built columns cover plates may be reduced in number and thickness, in the successive tiers, then omitted; then angles may be reduced in thickness and in length of legs, maintaining the same web plate or distance back to back of angles. (The distance back to back of angles is usually made ½ inch greater than the width of web plate.)

Reinforced Concrete Columns.

True reinforced concrete must of necessity be concrete reinforced or strengthened where the concrete is weak. Any system that combines steel and concrete where the steel is in compression is not reinforced concrete, but may be termed concrete-steel, a combination of the two materials assumed to be acting together. Concrete is strong in compression (confined or in short blocks), but weak in tension and shear. If steel is to reinforce concrete, it must do it by making up the lack existing in the concrete, that is, it must take up the tensile stresses and relieve the concrete of the same. There are tensile stresses in concrete acting as a simple post or column. This is scarcely recognized in books on engineering, though it is of tremendous importance, especially in reinforced concrete design. The cement mortar that is strongest in tension will make the strongest column. A bundle of thin straight wires would be useless as a column. But if the same wires were tightly bound about with a spiral wire, a heavy load could be borne by the same thin wires. Slender rods in a concrete shaft are very imperfectly and insecurely held together

and held from buckling by the concrete. Hence a concrete column built with slender rods in it, with the idea that these rods will reinforce it, is most absurdly designed. In spite of the fact that such design is standard and accepted by nearly all authorities on reinforced concrete, it is absolutely dangerous and indefensible. It has been the cause of a large number of very disastrous wrecks. Such design and practice cannot be too severely condemned. Books on reinforced concrete are woefully lacking and inexcusably blameworthy in this respect—that they encourage and hold out as standard and proper design such miserably poor construction. For a full presentation of this subject the reader is referred to the author's book "Concrete," to his paper, read before the American Society of Civil Engineers in March, 1910, entitled, "Some Mooted Questions in Reinforced Concrete Design," and to files of Engineering News and Concrete Engineering, 1907 to 1910, inclusive. No valid argument has been brought forth to controvert the author's position; tests and wrecks have amply demonstrated the soundness of it.

In this book only one form of reinforced concrete column will be considered as worthy of use, namely, the hooped column. A discussion of the proper dimensions of such a column will be found in the author's book, "Concrete." These are as follows:

Reinforced columns will be round or octagonal. They will have embedded in the concrete a coil of square steel having a diameter one-fortieth of the diameter of the column and eight upright rods just inside the coil and wired to the same, so as to prevent displacement of both coil and straight rods. The coil will have a diameter seven-eighths of that of the column and a pitch one-eighth of the diameter of the column. The upright rods will be of the same section as the rod in the coil. Where a coil ends, the next coil will lap one-half of a circle. Where upright rods end there will be a lap of 50 diameters of the steel rods.

On a column such as that described in the last paragraph a load per square inch may be allowed on the full section of the concrete, of 550 pounds, on columns having a length not more than ten times their diameter. Between 10 and 25 diameters the following load will be allowed:

$$p = 670 - 12\ l/D$$

where p = load per square inch,

l = length in inches,

D = diameter in inches.

Reinforced concrete columns should not be of greater length than 25 times their diameter.

Reinforced concrete cannot be recommended for economic construction in columns. Also the difficulties in the way of complete filling of the forms are many. Better construction is effected by the use of steel columns surrounded with concrete for fire protection or concrete columns in which are embedded stiff steel sections, which depend in a small degree only upon aid supplied by the concrete. These two classes of columns will be more fully described in what follows.

When steel columns are surrounded by concrete, to a depth of say 1½ or 2 inches over the metal, the steel columns should be designed in every respect as columns quite free of concrete, or as those protected by tile.

Efficient and safe columns can be made of steel angles or other stiff steel sections held together at intervals by batten plates riveted thereto, the whole being surrounded and filled with concrete. These batten plates should be sufficiently close so that each individual angle, or other stiff section of which the steel column is composed, will act as a short column between the batten plates. Such a steel column would not make a good compression member alone, but the concrete can be relied upon to add sufficient stiffness to the columns, within certain limits. Instead of battens, lattice may be used in the columns, except at girder connections, where angle shelves may be used upon which to rest the girders. The columns may be left open at girders for the passage of continuous rods.

It is recommended that concrete-steel columns such as those described in the preceeding paragraph be proportioned on the basis of a flat unit stress of 16,000 lbs. per sq. in., and that the width out to out of steel column be not less than one-twelfth of the unsupported height, and that concrete to a depth of 2 inches be used outside of all metal. The concrete should be considered merely as protecting the steel and carrying shear from one side to the other of the column. No compressive value should be allowed for the concrete.

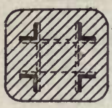


Fig. 12.

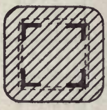


Fig. 13.

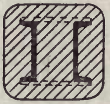


Fig. 14.

Fig. 15.

Figs. 12 to 15, inclusive, show examples of these concrete-steel columns. The dotted lines indicate batten plates or lattice bars. A good rule for the spacing of batten plates is to make them no farther apart than twelve times the width of the flange of the angle or channel.

TABLE IV.

Total Load in Thousands of Pounds, Allowed on Single Angles as Compression Members.

Size of Angle.	Unsupported Length of Member.						
	2 ft.	3 ft.	4 ft.	5 ft.	6 ft.	7 ft.	8 ft.
2 x2 x¼	11	9	8	6	..	..	..
2 x2 x⅜	16	13	11	9	..	..	..
2½x2½x¼	15	13	11	10	8	..	..
2½x2½x½	28	24	21	18	15	..	..
3 x3 x¼	18	17	15	13	12	10	..
3 x3 x⅝	43	39	35	31	27	22	..
3½x3½x⅜	33	30	28	25	23	20	18
3½x3½x¾	62	57	52	47	42	37	32
2½x2 x¼	13	11	9	7	..	..	..
2½x2 x½	24	20	17	14	..	..	..
3 x2½x¼	16	15	13	11	10	..	..
3 x2½x½	31	28	25	21	18	..	..
3½x2½x¼	18	16	14	13	11	9	..
3½x2½x⅝	42	38	33	29	25	20	..
3½x3 x⅜	30	27	25	22	19	17	14
3½x3 x¾	56	51	46	41	37	32	27
4 x3 x⅜	32	30	27	24	22	19	16
4 x3 x¾	61	56	51	46	41	36	31

TABLE V.

Total Loads in Thousands of Pounds, Allowed on Single Angles as Compression Members.

Size of Angle.	Unsupported Length of Member.						
	4 ft.	5 ft.	6 ft.	7 ft.	8 ft.	9 ft.	10 ft.
4 x4 x⅜	33	31	28	26	23	21	18
4 x4 x¾	63	58	53	48	43	38	34
6 x6 x⅜	56	54	51	48	46	43	41
6 x6 x¾	108	103	98	93	88	83	78
8 x8 x½	104	101	97	94	90	87	84
8 x8 x¾	154	149	143	138	133	128	123
5 x3 x⅜	31	28	25	22	19	..	..
5 x3 x¾	59	53	47	41	35	..	..
5 x3½x⅜	35	32	30	27	24	21	..
5 x3½x¾	67	61	56	51	45	40	..
6 x3½x⅜	40	37	33	30	27	24	..
6 x3½x¾	75	69	63	57	51	45	..
6 x4 x⅜	43	41	38	35	32	29	26
6 x4 x¾	83	77	72	66	61	55	49
8 x6 x½	88	85	81	77	74	70	66
8 x6 x¾	129	124	119	113	108	102	97

TABLE VI.

Total Load in Thousands of Pounds, Allowed on Two Angles Placed Thus  Seperated ½ in., as Compression Members.

Size of Angles.	Unsupported Length of Member.						
	4 ft.	5 ft.	6 ft.	7 ft.	8 ft.	9 ft	10 ft.
2½x2 x¼	25	23	21	19	17	15	..
2½x2 x½	46	42	39	35	31	27	..
3 x2½x¼	32	30	28	26	24	23	21
3 x2½x½	61	57	53	50	46	42	38
3½x2½x¼	37	35	33	31	29	28	26
3½x2½x½	70	66	63	59	56	52	49
3½x3 x⅜	58	55	52	49	46	43	41
3½x3 x¾	108	102	97	91	85	80	74
4 x3 x⅜	64	62	59	56	53	51	48
4 x3 x¾	121	116	111	105	100	94	89
5 x3 x⅜	74	71	68	65	62	59	56
5 x3 x¾	143	138	132	126	121	115	110
5 x3½x⅜	81	79	76	73	70	67	65
5 x3½x¾	156	151	145	140	135	130	124
6 x3½x⅜	91	87	84	81	77	74	71
6 x3½x¾	175	169	163	157	151	145	139
6 x4 x⅜	98	95	92	89	86	83	80
6 x4 x¾	189	183	178	172	167	161	156

TABLE VII.

Total Load in Thousands of Pounds, Allowed on Two Angles Placed Thus  as Compression Members.

Size of Angles.	Unsupported Length of Member.						
	4 ft.	5 ft.	6 ft.	7 ft.	8 ft.	9 ft	10 ft.
2½x2 x¼	22	20	17	15	..	..	..
2½x2 x½	41	36	31	26	..	..	..
3 x2½x¼	30	28	25	23	20	18	..
3 x2½x½	57	52	47	42	37	33	..
3½x2½x¼	33	30	27	24	22	19	..
3½x2½x½	62	56	51	45	40	34	..
3½x3 x⅜	56	52	49	45	41	38	34
3½x3 x¾	103	96	89	82	75	67	60
4 x3 x⅜	60	56	52	48	44	40	36
4 x3 x¾	112	104	96	88	80	73	65
5 x3 x⅜	68	63	59	54	49	44	40
5 x3 x¾	128	118	109	99	90	80	71
5 x3½x⅜	76	72	68	64	59	55	51
5 x3½x¾	144	135	127	119	111	102	94
6 x3½x⅜	85	80	75	70	65	61	56
6 x3½x¾	161	151	141	131	122	112	102
6 x4 x⅜	93	88	84	80	75	71	67
6 x4 x¾	176	168	159	151	142	133	125

TABLE VIII.

Total Load in Thousands of Pounds, Allowed on Two Angles Placed Thus  as Compression Members.

Size of Angles.	Unsupported Length of Member.						
	4 ft.	5 ft.	6 ft.	7 ft.	8 ft.	9 ft	10 ft.
2 x2 x¼	20	18	16	14	..	..	..
2 x2 x⅜	29	25	22	19	..	..	..
2½x2½x¼	28	25	23	21	19	..	..
2½x2½x½	51	47	43	39	35	..	..
3 x3 x¼	35	33	31	29	27	24	22
3 x3 x⅝	81	76	70	65	60	54	49
3½x3½x⅜	62	59	56	53	50	46	43
3½x3½x¾	117	111	105	98	92	86	79
4 x4 x⅜	74	71	68	64	61	58	55
4 x4 x¾	140	134	127	121	114	108	102
6 x6 x⅜	120	116	113	110	107	104	100
6 x6 x¾	231	224	218	212	205	199	192
8 x8 x½	218	214	210	205	201	197	192
8 x8 x¾	322	316	309	303	296	290	283

TABLE IX.

Total Load in Thousands of Pounds, on Four Angles Placed Thus  as Compression Members.

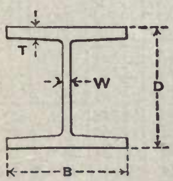
Size of Angles.	Unsupported Length of Member.						
	4 ft.	5 ft.	6 ft.	7 ft.	8 ft.	9 ft	10 ft.
2 x2 x¼	45	42	39	36	33	29	26
2 x2 x⅜	65	61	57	52	48	44	39
2½x2½x¼	60	57	53	50	47	44	41
2½x2½x½	114	108	103	97	91	86	80
3 x3 x¼	75	72	68	65	62	59	56
3 x3 x⅝	176	169	162	155	148	141	133
3½x3½x⅜	132	127	123	118	113	109	104
3½x3½x¾	251	243	234	226	217	209	200
4 x4 x⅜	155	150	145	141	136	131	126
4 x4 x¾	296	287	279	270	261	252	244
6 x6 x⅜	246	241	236	231	226	221	216
6 x6 x¾	476	467	458	449	439	430	421
8 x8 x½	445	439	432	426	419	413	406
8 x8 x¾	658	648	639	629	620	610	601

TABLE X.

Total Load in Thousands of Pounds, Allowed on Standard Gas Pipe as Compression Members.

Nominal Size of Pipe.	External Diam. in In.	Internal Diam. in In.	Unsup. Lgth. of Member. 5 ft.	6 ft.	7 ft.	8 ft.	9 ft.	10 ft.
2 in.	2.375	2.067	12	11	10	9	8	7
2½ in.	2.875	2.467	20	18	17	16	15	13
3 in.	3.500	3.066	27	26	25	23	22	21
3½ in.	4.000	3.548	34	32	31	30	28	27
4 in.	4.500	4.026	41	39	38	37	35	34
4½ in.	5.000	4.508	48	47	45	44	42	41
5 in.	5.563	5.045	58	56	54	53	51	50
6 in.	6.625	6.065	76	74	73	71	69	68
7 in.	7.625	7.023	96	94	92	90	89	87
8 in.	8.625	7.981	118	116	114	112	110	108
9 in.	9.625	8.937	142	140	138	135	133	131
10 in.	10.750	10.018	170	168	166	163	161	159

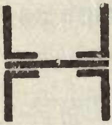
TABLE XI.



Total Load in Thousands of Pounds Allowed on Bethlehem H-Sections as Compression Members.

Dim. in In. and Weight of Sec. D	B	T	W	Weight in Lbs. per Ft.	Unsupported Lgth. of Member. 10 ft	12 ft	14 ft	16 ft	18 ft	20 ft.
8	8.00	½	.31	34.5	119	112	105	98	91	84
8½	8.16	¾	.47	53.0	184	173	163	152	142	132
8¾	8.24	⅞	.55	62.0	217	205	192	180	168	156
9	8.32	1	.63	71.5	251	237	223	209	196	182
9½	8.47	1¼	.78	90.5	320	302	285	268	251	234
10	10.00	⅝	.39	54.0	198	189	180	171	162	153
10¼	10.08	¾	.47	65.5	240	229	219	208	197	187
10½	10.16	⅞	.55	77.0	282	270	258	246	234	221
11	10.31	1⅛	.70	99.5	368	352	337	321	306	290
11½	10.47	1⅜	.86	123.5	457	438	420	401	381	363
11¾	11.92	⅝	.39	64.5	244	235	227	218	209	200
12	12.00	¾	.47	78.0	296	285	274	264	253	243
12½	12.16	1	.63	105.0	400	386	372	358	344	330
13	12.31	1¼	.78	132.5	506	488	471	453	436	418
13½	12.47	1½	.94	161.0	615	595	574	553	533	512
13⅞	13.96	¾	.47	91.0	353	343	332	321	311	300
14⅜	14.12	1	.63	122.5	477	463	449	435	421	407
15⅜	14.43	1½	.94	186.5	730	710	689	668	647	626
16⅜	14.74	2	1.25	253.0	994	966	939	911	884	856
16⅞	14.90	2¼	1.41	287.5	1130	1099	1069	1038	1007	976

TABLE XIII.



Total Load in Thousands of Pounds Allowed on I-Shaped Sections as Compression Members.

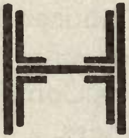
Web.	Angles.	Unsupported Length of Member. 10 ft.	12 ft.	14 ft.	16 ft.	18 ft.	20 ft.
12x5/16	3½x3 x5/16	115	103	91	79	...	...
12x5/16	3½x3 x½	164	149	134	119	104	...
12x½	3½x3 x½	186	168	151	133	116	...
12x5/16	4 x3 x5/16	131	120	109	99	88	77
12x5/16	4 x3 x½	186	173	159	146	132	118
12x½	4 x3 x½	210	194	178	162	146	130
12x5/16	5 x3 x5/16	158	149	140	131	122	113
12x5/16	5 x3 x½	226	214	203	191	179	167
12x½	5 x3 x½	252	238	225	211	198	184
12x⅜	6 x3½x⅜	227	217	207	197	187	177
12x⅜	6 x3½x¾	391	375	360	345	329	314
12x¾	6 x3½x¾	446	428	410	392	375	357
14x5/16	3½x3 x5/16	120	107	94	81	...	...
14x5/16	3½x3 x½	169	153	137	121	105	...
14x½	3½x3 x½	194	176	157	138	119	...
14x5/16	4 x3 x5/16	136	125	113	102	90	...
14x5/16	4 x3 x½	192	178	163	149	135	120
14x½	4 x3 x½	219	202	184	167	150	133
14x5/16	5 x3 x5/16	165	155	145	136	126	117
14x5/16	5 x3 x½	233	220	208	196	183	171
14x½	5 x3 x½	262	248	233	219	204	190
14x⅜	6 x3½x⅜	235	225	214	204	193	183
14x⅜	6 x3½x¾	399	383	368	352	336	320
14x¾	6 x3½x¾	463	444	425	406	387	368
14x⅜	6 x4 x⅜	244	232	221	210	199	188
14x⅜	6 x4 x¾	417	400	383	365	348	331
14x¾	6 x4 x¾	481	460	440	420	400	380
16x5/16	4 x3 x5/16	142	129	117	105	92	...
16x5/16	4 x3 x½	198	182	167	152	137	122
16x½	4 x3 x½	228	210	191	173	155	137
16x5/16	5 x3 x5/16	171	160	150	140	130	119
16x5/16	5 x3 x½	239	226	213	200	187	175
16x½	5 x3 x½	272	257	241	226	210	195
16x⅜	6 x3½x⅜	244	232	221	210	199	188
16x⅜	6 x3½x¾	408	391	375	358	342	326
16x¾	6 x3½x¾	480	460	440	420	400	380
16x⅜	6 x4 x⅜	252	240	228	217	205	193
16x⅜	6 x4 x¾	425	408	390	372	355	337
16x¾	6 x4 x¾	498	477	455	434	413	392

TABLE XII.

Total Load in Thousands of Pounds Allowed on I-Shaped Sections as Compression Members.

Web.	Angles.	Unsupported Length of Member. 8 ft.	10 ft.	12 ft.	14 ft.	16 ft.	18 ft.
6x¼	2½x2 x¼	56	48	41	...	...	...
6x¼	2½x2 x½	98	87	75	64	...	...
6x½	2½x2 x½	114	101	87	74	...	...
7x¼	2½x2 x¼	58	50	42	...	...	...
7x¼	2½x2 x½	100	88	76	64	...	...
7x½	2½x2 x½	118	104	90	76	...	...
8x¼	2½x2 x¼	60	51	42	...	...	...
8x¼	2½x2 x½	102	90	77	65	...	...
8x½	2½x2 x½	122	107	92	77	...	...
8x¼	3 x2½x¼	76	68	59	51	...	...
8x¼	3 x2½x½	132	119	106	94	81	...
8x½	3 x2½x½	153	138	124	109	94	...
9x5/16	3 x2½x5/16	98	87	77	66	...	...
9x5/16	3 x2½x½	140	126	112	99	85	...
9x½	3 x2½x½	158	142	126	111	95	...
9x5/16	3½x2½x5/16	113	103	93	84	74	65
9x5/16	3½x2½x½	160	148	136	123	111	98
9x½	3½x2½x½	179	165	151	137	123	109
9x5/16	3½x3 x5/16	118	108	97	87	76	...
9x5/16	3½x3 x½	170	156	143	129	115	101
9x½	3½x3 x½	189	174	158	143	127	112
9x5/16	4 x3 x5/16	132	123	113	104	94	85
9x5/16	4 x3 x½	190	178	165	153	140	128
9x½	4 x3 x½	210	196	182	168	154	140
9x5/16	5 x3 x5/16	156	148	140	132	124	116
9x5/16	5 x3 x½	227	216	205	195	184	173
9x½	5 x3 x½	248	236	224	212	199	187
10x5/16	3½x3 x5/16	121	110	99	88	77	...
10x5/16	3½x3 x½	173	159	145	131	117	103
10x½	3½x3 x½	194	178	162	146	130	114
10x5/16	4 x3 x5/16	135	125	115	105	96	86
10x5/16	4 x3 x½	194	181	168	155	142	129
10x½	4 x3 x½	215	200	186	171	156	142
10x5/16	5 x3 x5/16	160	152	143	135	127	118
10x5/16	5 x3 x½	231	220	209	197	186	175
10x½	5 x3 x½	254	241	228	216	203	191
10x⅜	6 x3½x⅜	228	219	209	200	191	182
10x⅜	6 x3½x¾	397	382	367	352	338	323
10x¾	6 x3½x¾	446	429	412	395	379	362

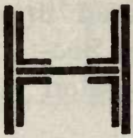
TABLE XIV.



Total Load in Thousands of Pounds Allowed on I-Shaped Sections as Compression Members.

Web.	Angles.	Cover Plates.	Area in sq. in.	L.R's of Gyr.	Unsup. Lgth. of M'b'r. 10 ft.	12 ft.	14 ft.	16 ft.	18 ft.	20 ft.
8x¼	3x2½x¼	8x¼	11.24	1.68	124	115	106	96	87	78
8x¼	3x2½x¼	8x½	15.24	1.86	175	163	152	140	129	118
8x¼	3x2½x½	8x½	20.00	1.78	226	210	195	179	163	148
8x½	3x2½x½	8x½	22.00	1.74	246	229	211	194	176	158
8x¼	3x2½x¼	9x¼	11.74	1.86	135	126	117	108	99	91
8x¼	3x2½x¼	9x½	16.24	2.09	193	182	171	160	150	139
8x¼	3x2½x½	9x½	21.00	1.97	245	230	215	201	186	171
8x½	3x2½x½	9x½	23.00	1.92	266	250	233	216	200	183
9x5/16	3x2½x5/16	9x5/16	14.92	1.86	171	160	149	137	126	115
9x5/16	3x2½x5/16	9x½	18.29	2.01	215	202	189	177	164	151
9x5/16	3x2½x½	9x½	21.81	1.94	253	238	222	206	191	175
9x½	3x2½x½	9x½	23.50	1.90	271	254	237	219	202	185
9x5/16	3x2½x5/16	10x5/16	15.54	2.05	183	173	162	152	141	131
9x5/16	3x2½x5/16	10x½	19.29	2.24	233	221	209	197	185	173
9x5/16	3x2½x½	10x½	22.81	2.15	273	258	243	229	214	199
9x½	3x2½x½	10x½	24.50	2.09	291	274	258	242	226	209
9x5/16	3½x3x5/16	9x5/16	16.16	1.91	187	175	163	151	140	128
9x5/16	3½x3x5/16	9x½	19.53	2.04	230	217	204	190	177	164
9x5/16	3½x3x½	9x½	23.81	1.99	279	262	245	229	212	195
9x½	3½x3x½	9x½	25.50	1.95	297	278	260	242	224	206
9x5/16	3½x3x5/16	10x5/16	16.78	2.08	199	188	176	165	154	143
9x5/16	3½x3x5/16	10x½	20.53	2.25	249	236	223	210	198	185
9x5/16	3½x3x½	10x½	24.81	2.17	298	282	266	250	234	218
9x½	3½x3x½	10x½	26.50	2.13	316	299	282	264	247	230
9x5/16	4x3 x5/16	9x5/16	16.80	2.02	198	186	174	163	151	140
9x5/16	4x3 x5/16	9x½	20.17	2.11	240	227	213	200	187	174
9x5/16	4x3 x½	9x½	24.81	2.10	295	279	262	246	229	213
9x½	4x3 x½	9x½	26.50	2.07	314	296	278	260	242	225
9x5/16	4x3 x5/16	10x5/16	17.42	2.17	209	198	187	175	164	153
9x5/16	4x3 x5/16	10x½	21.17	2.32	258	246	233	220	208	195
9x5/16	4x3 x½	10x½	25.81	2.26	313	297	281	265	249	233
9x½	4x3 x½	10x½	27.50	2.22	332	315	297	280	263	246
10x5/16	4x3 x5/16	10x5/16	17.74	2.15	212	201	189	178	166	155
10x5/16	4x3 x5/16	10x½	21.49	2.30	262	249	236	223	210	197
10x5/16	4x3 x½	10x½	26.13	2.25	316	300	284	268	252	236
10x½	4x3 x½	10x½	28.00	2.20	337	319	302	284	266	248
10x5/16	4x3 x5/16	11x5/16	18.36	2.33	224	213	202	191	180	169
10x5/16	4x3 x5/16	11x½	22.49	2.50	279	267	254	242	229	217
10x5/16	4x3 x½	11x½	27.13	2.43	335	319	304	288	273	257
10x½	4x3 x½	11x½	29.00	2.37	356	339	322	305	287	270

TABLE XV.



Total Load in Thousands of Pounds Allowed on I-Shaped Sections as Compression Members.

Web.	Angles.	Cover Plates.	Area in sq. in.	L.R's of Gyr.	Unsup. Lgth. of M'b'r. 10 ft.	12 ft.	14 ft.	16 ft.	18 ft.	20 ft.
10x⅜	6x3½x⅜	13x⅜	27.18	3.07	351	339	327	315	302	290
10x⅜	6x3½x⅜	13x¾	36.93	3.27	483	467	451	436	420	404
10x⅜	6x3½x¾	13x¾	49.49	3.22	645	624	603	581	560	538
10x½	6x3½x¾	13x¾	50.74	3.21	661	639	617	595	573	551
10x⅜	6x3½x⅜	14x⅜	27.93	3.23	364	352	340	328	316	304
10x⅜	6x3½x⅜	14x¾	38.43	3.47	507	492	476	461	445	430
10x⅜	6x3½x¾	14x¾	50.99	3.38	670	649	628	607	586	565
10x½	6x3½x¾	14x¾	52.24	3.36	686	664	643	621	599	578
12x5/16	4x3 x5/16	12x5/16	19.61	2.48	243	232	221	210	199	188
12x5/16	4x3 x5/16	12x½	24.11	2.69	304	292	279	267	254	242
12x5/16	4x3 x½	12x½	28.75	2.59	360	344	329	314	298	283
12x½	4x3 x½	12x½	31.00	2.52	386	368	351	334	317	300
12x5/16	4x3 x5/16	13x5/16	20.24	2.67	255	244	234	223	213	202
12x5/16	4x3 x5/16	13x½	25.11	2.91	322	310	298	286	274	262
12x5/16	4x3 x½	13x½	29.75	2.79	378	363	348	333	319	304
12x½	4x3 x½	13x½	32.00	2.71	404	388	371	355	338	322
12x⅜	6x3½x⅜	13x⅜	27.93	3.03	360	348	335	322	309	296
12x⅜	6x3½x⅜	13x¾	37.68	3.23	492	475	459	443	427	410
12x⅜	6x3½x¾	13x¾	50.24	3.20	655	633	611	589	567	545
12x½	6x3½x¾	13x¾	51.74	3.18	673	651	628	605	583	560
12x⅜	6x3½x⅜	14x⅜	28.68	3.18	373	361	348	336	323	310
12x⅜	6x3½x⅜	14x¾	39.18	3.43	516	500	484	468	453	437
12x⅜	6x3½x¾	14x¾	51.74	3.36	679	658	636	615	594	572
12x½	6x3½x¾	14x¾	53.24	3.33	698	676	653	631	609	587
14x5/16	4x3 x5/16	14x5/16	21.49	2.84	274	263	253	242	232	221
14x5/16	4x3 x5/16	14x½	26.74	3.11	347	335	323	311	299	287
14x5/16	4x3 x½	14x½	31.38	2.97	404	389	374	359	345	330
14x½	4x3 x½	14x½	34.00	2.88	435	418	402	385	369	352
14x5/16	4x3 x5/16	15x5/16	22.11	3.05	286	276	265	255	245	235
14x5/16	4x3 x5/16	15x½	27.74	3.35	364	353	341	329	318	306
14x5/16	4x3 x½	15x½	32.38	3.19	422	407	393	379	365	351
14x½	4x3 x½	15x½	35.00	3.09	453	437	422	406	390	374
14x⅜	6x3½x⅜	14x⅜	29.43	3.14	382	369	356	343	330	317
14x⅜	6x3½x⅜	14x¾	39.93	3.40	525	509	493	476	460	444
14x⅜	6x3½x¾	14x¾	52.49	3.33	688	666	644	622	600	578
14x½	6x3½x¾	14x¾	54.24	3.30	710	687	664	641	618	596
14x⅜	6x3½x⅜	15x⅜	30.18	3.31	395	383	370	357	345	332
14x⅜	6x3½x⅜	15x¾	41.43	3.61	550	534	518	502	486	470
14x⅜	6x3½x¾	15x¾	53.99	3.50	713	692	670	649	627	606
14x½	6x3½x¾	15x¾	55.74	3.47	735	713	691	668	646	624

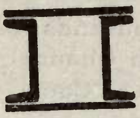
TABLE XVI.

Total Load in Thousands of Pounds Allowed in Latticed Channel Sections as Compression Members.

D not less than .65 of the depth of channel.

Size of Channels.		Unsupported Length of Member.					
Depth in In.	Weight in Lbs. per Foot.	10 ft.	12 ft.	14 ft.	16 ft.	18 ft.	20 ft.
5	6.5	45	43	40	37	34	31
5	**9.0**	60	56	52	48	44	40
5	**11.5**	76	70	65	60	54	49
6	**8.**	58	55	53	50	47	44
6	**10.5**	74	71	67	63	59	55
6	**13.0**	91	86	81	76	71	66
6	**15.5**	108	102	96	90	83	77
7	**9.75**	72	69	66	63	60	57
7	**12.25**	90	86	82	78	75	71
7	**14.75**	108	103	98	93	88	84
7	**17.25**	125	119	114	108	102	96
7	**19.75**	143	136	129	122	116	109
8	11.25	87	84	81	78	75	72
8	13.75	104	100	96	93	89	85
8	16.25	122	118	113	108	104	99
8	18.75	140	135	129	124	119	113
8	21.25	159	152	146	140	133	127
9	13.25	103	100	97	93	90	87
9	15.00	116	112	109	105	102	98
9	20.00	153	148	143	138	133	128
9	25.00	190	184	177	171	164	157
10	15.00	120	116	113	110	107	103
10	20.00	156	152	147	143	138	134
10	25.00	194	189	183	177	171	165
10	30.00	232	225	218	211	204	196
10	35.00	270	261	253	244	236	227
12	20.50	165	161	158	154	151	147
12	25.00	200	196	191	186	182	177
12	30.00	239	234	228	222	216	211
12	35.00	278	272	265	258	251	244
12	40.00	317	309	301	293	285	277
15	33.00	277	272	267	262	257	252
15	35.00	287	282	277	272	267	261
15	40.00	327	321	315	309	303	297
15	45.00	368	361	354	347	340	333
15	50.00	408	400	392	384	377	369
15	55.00	448	439	431	422	413	405

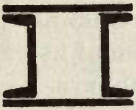
TABLE XVII.



Total Load in Thousands of Pounds Allowed on Channel and Plate Sections as Compression Members.

Size of Channels			Unsupported Length of Member.					
Depth in In.	Weight in Lbs. per Ft.	Size of Plates.	10 ft.	12 ft.	14 ft.	16 ft.	18 ft.	20 ft.
7	9.75	9x¼	128	123	118	112	107	102
7	9.75	9x½	185	177	169	161	154	146
7	12.25	9x5/16	161	154	147	140	133	126
7	12.25	9x9/16	217	208	198	189	180	170
7	14.75	9x⅜	192	184	175	167	158	150
7	14.75	9x⅝	248	238	227	216	205	194
7	17.25	9x7/16	224	213	203	193	183	173
7	17.25	9x11/16	280	267	255	242	230	217
7	9.75	11x¼	146	141	136	131	126	122
7	9.75	11x½	219	212	205	198	192	185
7	12.25	11x5/16	183	177	171	164	158	152
7	12.25	11x9/16	257	249	240	232	224	216
7	14.75	11x⅜	220	212	205	197	190	182
7	14.75	11x⅝	294	284	275	265	256	246
7	17.25	11x7/16	257	248	239	230	221	213
7	17.25	11x11/16	330	319	309	298	287	276
8	11.25	10x¼	151	146	140	135	129	124
8	11.25	10x½	215	207	199	192	184	176
8	13.75	10x5/16	184	177	171	164	157	151
8	13.75	10x9/16	248	239	230	221	212	203
8	16.25	10x⅜	219	211	202	194	186	178
8	16.25	10x⅝	283	272	261	251	240	230
8	18.75	10x7/16	253	243	233	224	214	205
8	18.75	10x11/16	317	305	293	281	269	257
8	11.25	12x¼	169	164	159	154	149	144
8	11.25	12x½	249	242	235	228	221	214
8	13.75	12x5/16	207	201	195	189	183	177
8	13.75	12x9/16	287	279	271	263	255	246
8	16.25	12x⅜	246	239	232	224	217	210
8	16.25	12x⅝	327	317	308	298	289	280
8	18.75	12x7/16	285	277	268	260	252	243
8	18.75	12x11/16	366	355	344	334	323	313

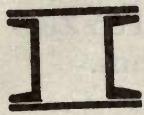
TABLE XVIII.



Total Load in Thousands of Pounds Allowed on Channel and Plate Sections as Compression Members.

Size of Channels			Unsupported Length of Member.					
Depth in In.	Weight in Lbs. per Ft.	Size of Plates.	10 ft.	12 ft.	14 ft.	16 ft.	18 ft.	20 ft.
9	13.25	11x¼	175	169	164	158	153	147
9	13.25	11x½	246	238	231	223	215	207
9	15.00	11x5/16	206	199	193	186	180	173
9	15.00	11x9/16	278	269	260	251	242	233
9	20.00	11x⅜	261	253	244	235	227	218
9	20.00	11x⅝	333	322	311	300	289	278
9	25.00	11x7/16	316	306	295	284	274	263
9	25.00	11x11/16	388	375	362	349	336	322
9	13.25	13x¼	192	188	183	178	173	168
9	13.25	13x½	280	273	266	259	252	245
9	15.00	13x5/16	229	223	217	211	205	199
9	15.00	13x9/16	316	308	300	292	284	275
9	20.00	13x⅜	289	281	274	266	259	251
9	20.00	13x⅝	377	367	357	347	338	328
9	25.00	13x7/16	350	340	331	322	312	303
9	25.00	13x11/16	438	426	414	403	391	380
10	15	12x5/16	219	213	207	201	195	189
10	15	12x9/16	298	290	281	273	264	256
10	20	12x⅜	276	268	260	252	244	236
10	20	12x⅝	355	345	335	324	314	304
10	25	12x7/16	334	324	314	305	295	285
10	25	12x11/16	413	401	389	377	364	352
10	30	12x½	392	380	368	356	345	333
10	30	12x¾	471	457	443	428	414	400
10	15	14x5/16	241	236	230	225	220	214
10	15	14x9/16	336	328	320	312	305	297
10	20	14x⅜	304	297	290	283	276	269
10	20	14x⅝	398	389	379	370	360	351
10	25	14x7/16	367	358	350	341	333	324
10	25	14x11/16	461	450	439	428	417	406
10	30	14x½	430	420	410	399	389	379
10	30	14x¾	524	512	499	487	474	461

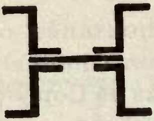
TABLE XIX.



Total Load in Thousands of Pounds Allowed on Channel and Plate Sections as Compression Members.

Size of Channels: Depth in In.	Size of Channels: Weight in Lbs. per Ft.	Size of Plates.	Unsupported Length of Member. 10 ft.	12 ft.	14 ft.	16 ft.	18 ft.	20 ft.
12	20.5	14x3/8	307	300	293	286	278	271
12	20.5	14x5/8	401	392	382	373	363	354
12	25	14x7/16	366	357	349	340	331	323
12	25	14x11/16	460	449	438	427	416	405
12	30	14x1/2	429	419	408	398	387	377
12	30	14x3/4	523	510	498	485	472	459
12	35	14x9/16	492	480	468	456	443	431
12	35	14x13/16	586	572	557	543	528	514
12	20.5	16x3/8	333	327	320	314	307	301
12	20.5	16x5/8	443	434	425	416	407	399
12	25	16x7/16	397	389	381	373	366	358
12	25	16x11/16	507	496	486	476	466	456
12	30	16x1/2	465	456	446	437	428	418
12	30	16x3/4	574	563	551	539	528	516
12	35	16x9/16	533	522	511	500	489	479
12	35	16x13/16	642	629	616	602	589	576
15	33	17x7/16	483	475	466	457	448	439
15	33	17x11/16	601	589	578	567	556	545
15	40	17x1/2	564	553	543	533	522	512
15	40	17x3/4	681	668	656	643	630	617
15	45	17x9/16	634	622	610	599	587	575
15	45	17x13/16	751	737	723	709	695	680
15	50	17x5/8	704	690	677	664	651	637
15	50	17x7/8	821	805	790	774	758	743
15	33	19x7/16	513	505	497	489	481	473
15	33	19x11/16	646	636	625	615	604	594
15	40	19x1/2	599	589	580	570	561	551
15	40	19x3/4	731	719	708	696	684	672
15	45	19x9/16	674	663	652	641	630	619
15	45	19x13/16	806	793	780	767	753	740
15	50	19x5/8	748	736	724	711	699	687
15	50	19x7/8	880	866	851	837	822	808

TABLE XX.



Total Load in Thousands of Pounds Allowed on Zee-Bar Columns.

Width of Web Plate in In.	Web of Z-Bar in In.	Thickness of Metal.	Unsupported Length of Column. 10 ft.	12 ft.	14 ft.	16 ft.	18 ft.	20 ft.
6	3	1/4	107	100	93	86	79	*72*
6	3	5/16	136	128	119	110	102	*93*
6	3	3/8	157	147	137	127	116	*106*
6	3	7/16	186	174	163	151	139	*128*
6	3	1/2	205	192	178	165	152	*139*
6	3	9/16	234	219	205	191	176	*162*
7	4	5/16	141	134	128	122	115	109
7	4	1/4	178	170	162	154	146	138
7	4	3/8	215	206	196	187	178	168
7	4	7/16	239	228	217	206	195	185
7	4	1/2	276	263	251	239	227	215
7	4	9/16	313	299	286	272	259	245
7	4	5/8	330	316	301	286	271	257
7	4	11/16	367	351	335	319	303	287
7	4	3/4	404	387	370	353	336	319
7	5	5/16	204	197	190	183	176	169
7	5	3/8	247	238	230	222	213	205
7	5	7/16	290	280	271	261	251	241
7	5	1/2	317	306	295	284	273	262
7	5	9/16	360	348	335	323	311	299
7	5	5/8	403	390	376	363	350	336
7	5	11/16	424	409	395	380	366	351
7	5	3/4	466	450	435	419	403	388
7	5	13/16	509	492	476	459	442	425
8	6	3/8	284	276	268	260	252	244
8	6	7/16	334	325	315	306	297	287
8	6	1/2	384	373	363	352	342	331
8	6	9/16	416	404	392	381	369	357
8	6	5/8	465	452	439	426	413	401
8	6	11/16	515	501	486	473	458	444
8	6	3/4	540	525	510	495	479	464
8	6	13/16	589	572	556	540	523	507
8	6	7/8	636	618	601	583	565	547

CHAPTER V.

Lintels.

Some examples of cast iron lintels will be found in Chapter VI. This chapter will be taken up with the subject of steel lintels.

Lintels are often made so that only the edge of a plate or the edge of an angle will show in the face of the wall. The steel must be set back a little so as to allow pointing at the supports.

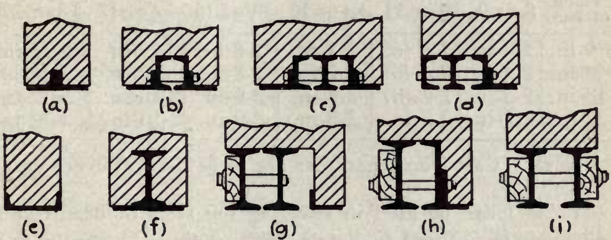


Fig. 1

Fig. 1 shows a number of different styles of lintels.

In a solid wall it is usual to calculate the lintels as carrying a height of wall equal to one-third of the opening. Where the top of a wall or a large opening occurs a short distance above the lintel, say a height equal to the span or less, the full height of wall should be borne by the lintel. If floor concentrations or wall piers occur over the opening, the effect of these loads must be considered.

A lintel such as that shown at (a) Fig. 1, made up of 2 4″x3″x5-16″ angles with the short legs vertical and riveted together, may be used in a 9-in. solid wall for spans up to 8 ft. Two 6″x3½″x⅜″ angles may be used in like manner in a 13-in. wall for spans up to 8 ft.

A standard 9-in. channel may be used as at (e) for openings up to 6½ ft., and a standard 12-in. channel may

be used for openings up to 7½ ft., these in 9-in. and 13-in. walls, respectively.

In the following table, taken from "Steel in Construction" (Pencoid Iron Works), the lintels are selected to deflect 1/360 of the span up to 10 ft., and 1/500 of the span above 15 ft. The fiber stress, assuming the lintel to carry a height of wall ⅓ of the opening, is within 16,000 lbs. per sq. in.

TABLE I.

SIZE OF STANDARD I-BEAMS FOR LINTELS.

Span in Feet.

Thick. of Wall	8 or 9	10 or 11	12 or 13	14 or 15	16 or 17	18 or 20
9-in.	2-4-in.	2-5-in.	2-7-in.	2-8-in.	2-9-in.	2-12-in.
13-in.	2-4-in.	2-6-in.	2-7-in.	2-8-in.	2-9-in.	2-12-in.
18-in.	2-5-in.	2-7-in.	2-8-in.	2-9-in.	2-10-in.	2-12-in.
22-in.	2-5-in.	2-7-in.	2-8-in.	2-9-in.	2-10-in.	2-12-in.

NOTE: Cast iron separators are to be used in every case.

Table I can be used in selecting the sizes of beams and channels to be used in lintels, such as those shown in (c) and (d) Fig. 1, using two channels in place of a beam. The angles should be counted as simply acting as supports for the first few courses of bricks. The methods given in Chapter VI may be employed to find the size of beams and channels required in any lintel.

Sometimes a loose angle is used with the lintel, as at (g) Fig. 1. This is merely to carry the face brick up to the level of the top of the beams. Sometimes an I-beam, instead of the channel shown at (d), is exposed in the face of the wall; this allows building up of the brick work over supports, if no offset occurs in the wall at jambs. The wooden pieces shown at (g), (h) and (i) are for nailing on the wood finish. Separator bolts should be ordered long enough to include these.

Separators for lintels are usually short pieces of gas pipe slipped over the bolts.

CHAPTER VI.

Beams.

Beams may be made of wood, cast iron, steel or reinforced concrete, though cast iron is seldom used for beams, except in the case of window lintels and the like.

The selection of wooden beams or joists to carry a certain load is restricted, and is also simplified by the commercial sizes. A unit stress of 800 lbs. per sq. in. should be used for soft woods, such as white pine, and 1,000 lbs. may be used for white oak and long-leaf yellow pine. A simple way to find the size of wooden beam is by use of Table I. In this table the coefficient C is equal to the product of the span of the beam in feet and the total uniform load in pounds, which the beam can safely carry. If, for example, a wooden beam of a span of 10 feet is to carry a load of 150 lbs. per lineal foot, or 1,500 lbs., the value of the coefficient C for such a beam would need to be 10x1,500 or 15,000. A 2x10 beam in white pine or a 2x9 beam in oak or yellow pine would suffice. The total load, uniformly distributed, that any beam may safely carry is readily found from the table by dividing the value C by the span of the beam in feet. (Note that C is the product of one-ninth of the unit stress by the width of beam by the square of the depth.)

If the load on a beam is central and concentrated, instead of being uniformly distributed, it should be doubled for finding the size required, as such a concentrated load is twice as effective in producing bending moments as the same load uniformly distributed. If, for example, a wooden girder having a span of 16 feet is to carry a center load of 3,500 lbs., the value of C would be 2x3,500x 16=112,000. A yellow pine beam 4x16 would suffice.

The depth of wooden beams should generally be between one-tenth and one-twentieth of the span. Beams deeper than one-tenth will be overstressed in shear, when

strained to their capacity in bending, with a uniform load; beams shallower than about one-twentieth will deflect too much under load.

TABLE I.

Size of Beam in Inches.	C, for White Pine.	C, for Oak or Y. P.
2x 4	2,840	3,550
2x 5	4,440	5,550
2x 6	6,400	8,000
2x 8	11,400	14,220
2x 9	14,400	18,000
2x10	17,800	22,220
2x12	25,600	32,000
3x14	52,270	65,330
3x15	60,000	75,000
4x16	91,020	113,770

Cast Iron Beams.

Cast iron can only be used economically in beams in shapes that have wide or heavy tension flanges, because of the weakness of cast iron in tension.

(a) (b) (c) (d)

Fig. 1.

Lintels for brick walls are sometimes made in cast iron in shapes such as shown in Fig. 1. Calculating 4,000 lbs. per sq. in. tension on the cast iron, and assuming that a height of wall one-third the height of the opening is carried by the lintel, the lintel shown in end view at (b) could be used in a 9-in. brick wall, in ½-in. metal, for openings up to about five feet. In ¾-in. metal it could be used for openings up to six feet. The lintel shown in end view at (d) has just about double the strength and double the load of that shown at (b), so that the same limits

of spans can be used. The one shown at (c) could span larger openings, theoretically, but it is not advisable to use long beams in cast iron, because of the uncertainties in the metal. Steel beams are more reliable for large openings. Also, if the opening has a pier or a concentrated load above it, steel lintels should be used, designed to carry that load.

Steel Beams.

Nearly all of the beams in a building are designed for uniform load, so that the determination of the sizes is generally a simple matter, when tables are at hand. It is a common standard in building work to allow 16,000 lbs. per sq. in. extreme fiber stress on the steel. This is a correct unit for quiescent loads, such as those in buildings. It would be too high for rolling loads such as bridges, so that the methods and units of this chapter cannot be employed to design bridges. It should be clearly understood that this chapter, and in fact this entire book, applies only to building work. Bridges are designed on quite a different standard and by different methods.

Many handbooks give tables showing the total load which a beam will carry. The tables of this chapter give instead a quantity for the several sizes of beams, designated Q, by which the capacity of a beam may readily be found. The quantity Q is equal to the product of the span of a beam in feet and the load in tons (of 2,000 lbs.) that the beam can safely carry as a uniformly distributed load.

To find the size of a beam for a given case, it is only necessary to find the load in tons that the beam must carry and multiply this by the span. Then by looking in the tables find a value Q that equals this product. That beam is then a proper size for the case, assuming that it is held against lateral displacement in the building.

The full strength of beams, as exhibited in this chapter, is only realized when the beams are properly stiffened and properly supported at the ends. For the end supports of beams see Chapter X. The matter of stiff-

ening of the beams or lateral support will be considered here, as this is a matter vitally connected with the general strength of the beam, and it is a matter not so generally understood nor appreciated as that of the necessity for proper support at the ends of a beam.

In *Engineering News,* January 6, 1910, will be found the record of an experiment on small beams built of tin plate, in which the mere addition of end stiffeners to one of two beams identically made added 129 per cent to the ultimate strength of the beam thus stiffened. The purpose of adding the stiffeners was not to prevent the web from buckling, but to prevent the beam from keeling over at the support. The beam which had not the end stiffeners failed by leaning of the web in opposite directions at the ends, or by a twisting of the entire beam. This shows conclusively, what analysis would dictate, namely, that it is necessary in all beams, in order to develop the full strength, that the beam be held against lateral tilting at the ends. In the ordinary case in buildings this is accomplished by building the ends of beams into the wall or by the riveted end connections of the beam.

The top flange of a beam should also be held laterally at intermediate points. This is usually accomplished by the arches between the beams or the floor slabs resting on top of them. Where it is not practicable to stiffen the compression flange of a beam continuously, it should be braced at intervals. The intervals should not be more than about sixteen times the width of the flange, if the full tabular value of the beam is used. If it is necessary to have the compression flange unsupported for 50 times the width, only one-half of the tabular value for the strength of the beam should be used. At 25 times the flange width, unsupported, use 7/8 of the tabular load; at 33 times, use 3/4; at 42 times, use 5/8.

When there is any plastered work or concrete covering, the depth of steel beam should not be less than about one-twenty-fourth of the span, so that the deflection will not be too great. In other work a limit of one-thirtieth may be observed.

Examples.

(1) Given a mill roof with channel purlins spaced 5 ft. apart, 2-inch matched tongue-and-groove board sheathing, tar and gravel covering, snow load 50 lbs. per sq. ft., span between trusses 16 ft. Assume 7 lbs. per sq. ft. for covering, 8 lbs. per sq. ft. for sheathing, and 3 lbs. per sq. ft. for purlins. The load per foot on purlin is $(50+7+8+3)\times5=340$ lbs. The load carried by one purlin is $340\times16=5{,}440$ lbs., or 2.72 tons. Q is then $2.72\times16=43.5$. Q for a standard 8-in. 11¼ lb. channel is 43.2, hence this size would be used.

(2) Given floor beams supporting tile arches, span 13 ft., distance apart 6 ft., 1″ floor on sleepers filled with cinder concrete, live load 80 lbs. per sq. ft. Assume 10-in. arches, which weigh 39 lbs. per sq. ft. The several weights are: 15 lbs. for cinder concrete and sleepers; 4 lbs. for wooden flooring; 7 lbs. for steel, fireproofing and ceiling, and 80 lbs. for live load. This is a total of 145 lbs. per sq. ft. or 7.83 tons per beam. Q is $7.83\times18=140.9$. By interpolating between a 10″ beam 25 lbs. and 40 lbs., it is found that a 10″ 30 lb. beam would suffice. If standard beams are preferred 12″ 31½ lb. beams could be used. Ten-inch arches can be used on these by offsetting the ceiling at each beam. If conditions permitted, closer spacing of the beams could be used, and 10-in. 25 lb. I-beams would suffice.

(3) Given floor beams spaced 9 ft. apart supporting a 4-inch reinforced concrete slab with 1-inch tile floor on the same, the span being 20 ft., and live load 100 lbs. per sq. ft. The load per sq. ft. is as follows: Live load 100, concrete 50, tile and filling 20. This is 1,530 lbs. per lineal foot of beam. Adding for weight of beam and surrounding concrete 150 lbs. per ft., the weight on the beam is $1{,}680\times20=33{,}600$, or 16.8 tons. Q is 336. By interpolation a 15″ 50-lb. beam is found to be correct.

(4) Given a double wall beam to be made up of an I-beam and a channel of the same depth, the beam to carry 12 ft. of vertical height of a 13-inch wall and 4 ft. of a

floor load at 200 lbs. per sq. ft. total, the span being 18 ft. The wall will weigh 130×12 or 1,560 lbs. per lin. ft. Adding to this 800 lbs. for the floor load and 60 lbs. for the weight of the beam, we have 2,420 lbs. per lineal foot. The load carried by the beam is then 21.78 tons, and Q is 392. By trial it is seen that a 12″ I 40 lbs. and a 12″ channel 35 lbs. will have a combined value of Q equal to this. By using a channel and beam of different depths standard sections could be employed, as a 15″ beam 42 lbs. and a 12″ channel 20.5 lbs.

(5) Given a system of T bars, supporting 18-inch book tiles, carried on purlins spaced 10 ft. apart. Weight of book tile and roofing per sq. ft. 30 lbs., live load 50 lbs. Total weight on T bar $80 \times 1\frac{1}{2} \times 10 = 1{,}200 = 0.6$ ton. $Q = 6$. A $3 \times 3 \times 10.1$ lb. T would suffice.

(6) Given a system of double angles spaced 4 ft. apart on a span of 8 ft. supporting a balcony; live load 60 lbs. per sq. ft. Assume a slab weight of 50 lbs. per sq. ft. total. A pair of angles will carry 110x4x8=3,520 lbs or 1.76 tons. Q=1.76x8=14.08. The value of Q for 2 angles 4″x3″x⅜, long legs vertical, is 15.6. Note that these angles would weigh more than beams or channels of the same strength, and they would hence not be the most economical section to use. However, they afford a better seat for a slab, if it is the intention to keep the supporting beam within the depth of the slab. Note that a 4x5x15.7-lb. T-bar would be of sufficient strength for this case, but the 5-inch stem might be too deep.

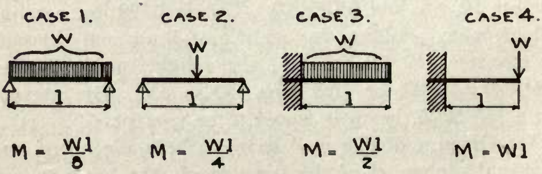


Fig. 2.

In Fig. 2 are given several cases of beams and the bending moments for each. Case 1 is that of a simple beam uniformly loaded. The values of Q in Tables II to VII are for this case. They can be made to apply to any of the other cases as follows:

For a single concentrated load at the center of a given span it is seen that the bending moment M is just double that which the same load would produce if uniformly distributed over the span. Hence a single concentration at the center of a span will give the same bending as twice that load uniformly distributed. To use Tables II to VII, then, we will have to double the concentration and use that load as a uniform load.

Examples:

(1) Given an I-beam on a 12-ft. span supporting a concentrated load at the center of span of 24,000 lbs. Doubling this to find the equivalent uniform load and multiplying by 12 (after reducing to tons) we have 288 as the value of Q. A 15″-I, 42 lbs. would then be required.

(2) Given a pair of beams on a span of 8 ft. supporting a column load at the center of span of 150,000 lbs. $150{,}000 \times 2 = 300{,}000$ lbs. or 150 tons. $150 \times 8 = 1{,}200$, the value of Q. Two 20″ beams 65 lbs. have a value $Q = 1{,}248$.

(3) Given an opening in an 18-in. wall 17 ft. wide and a floor-girder just above the middle of same with a load of 37,500 lbs. Call the span of the lintel 18 ft. wide and as sume a wall load 6 ft. high or $180 \times 6 \times 18 = 19{,}440$ lbs. The equivalent uniform load is $37{,}500 \times 2 + 19{,}440 = 94{,}440$ lbs. or 47.22 tons. Q is $47.22 \times 18 = 850$. Two 18″ beams 55 lbs. would be somewhat stronger than necessary.

When the concentrated load is not at the center of span, a special case arises, and the simplified methods of this chapter do not apply.

Case 3 in Fig. 2 is for a cantilever beam uniformly loaded. It is seen that the bending moment is four times as great for the same load and span as that found in a

simple beam. Hence the equivalent load for a simple span to be used in tables II to VII will be four times the actual uniform load on the span.

Examples:

(1) Given a roof truss load of 80,000 lbs. to be distributed by means of two wall beams 5 ft. long into a brick wall.

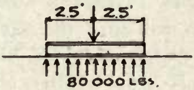


Fig. 3.

Here the load which is uniformly distributed is an upward one, being the reaction of the brick wall against the beams. The span l is 2.5 ft. and the load on each of these cantilevers is 40,000 lbs. The equivalent load for a simple beam is $4 \times 40{,}000 = 160{,}000$ lbs. or 80 tons. Q is $80 \times 2\frac{1}{2} = 200$. Two 9″ beams 21 lbs. have a value of Q equal to 201.6.

(2) Given a building in which the wall is omitted at the corner for a distance of 6 feet, there being no corner post but cantilever beams at the second story meeting at the corner and supporting the wall and floors above. Assume that the total weight of wall and floor load is 4,200 lbs. per running foot, and that the effectual span of the cantilever is 7 ft. The load carried by the cantilever is $4{,}200 \times 7 = 29{,}400$ lbs. The equivalent load for a simple span is $29{,}400 \times 4 = 117{,}600$ lbs. or 58.8 tons. Q is $58.8 \times 7 = 411.6$. Two 12-in. 35-lb. beams would come within a small percentage of filling the requirements.

(3) Given roof rafters projecting 4 ft. beyond a wall and supporting rienforced concrete slabs, the rafters being spaced 6 ft. and the total load carried being 80 lbs. per horizontal square foot. The load on a rafter is $80 \times 4 \times 6 = 1{,}920$ lbs. The equivalent load for a simple beam is $1{,}920 \times 4 = 7{,}680 = 3.84$ tons. Q is $3.84 \times 4 = 15.36$.

The rafter could be a 4″ 7½-lb. beam, or a 5″ 6½-lb. channel, or a 2-4″×3″×⅜″ angles, or a 4″×¼″ zee-bar, or a 4″×5″×16.7-lb. T-bar.

Case 4 in Fig. 2 is for a concentrated load at the end of a cantilever. The moment here is eight times as great for a given span and load as that for a simple beam. The equivalent load for a simple beam is then eight times the amount of the concentration.

Examples.

(1) Given a balcony 7 ft. wide supported on cantilever beams spaced 12 ft. apart. A facia beam supports one side of a slab and a railing. Assume the floor load on the facia beam to be 420 lbs. per ft., and the railing to weigh 40 lbs. per ft. The concentrated load at the end of the cantilever beam is then 460×12=5,520 lbs. or 2.76 tons. The equivalent uniform load for a simple span is 2.76×8=22.08. Q is 22.08×7=154.56. This could be a 10″-I 35 lbs. or 2-10″ channels 20 lbs.

(2) Given a cantilever beam supporting a column at its end, the overhang being 4 ft and the column load being 120,000 lbs. The equivalent load for a simple beam is 120,000×8=960,000, or 480 tons. Q is 480×4=1,920. This would require 2-24″ I beams 85 lbs.

(3) Given a cantilever beam supporting a uniform load of 800 lbs. per ft. and a concentrated load at the outer end of 10,000 lbs., the span being 10 ft. The equivalent uniform load for a simple beam is 8,000 (the total uniform load) ×4+10,000×8 or 112,000 lbs.=56 tons. Q is 56×10=560. A 15″ I 80 lbs. would do, but a 20″ I 65 lbs. is much stronger and would weigh less.

TABLE II.

Capacity of Standard I Beams and Channels

Extreme fiber stress 16,000 lbs. per sq. in.

Size.	Q.	Size.	Q.	Size.	Q.
24″ I 100 lb.	1058.2	10″ I 40 lb.	169.1	12″ Ch. 40 lb.	174.
24″ I 80 lb.	928.0	10″ I 25 lb.	130.1	12″ Ch. 20½ lb.	114.
20″ I 100 lb.	883.2	9″ I 35 lb.	132.3	10″ Ch. 35 lb.	123.
20″ I 80 lb.	782.4	9″ I 21 lb.	100.8	10″ Ch. 15 lb.	71.
20″ I 75 lb.	676.9	8″ I 25½ lb.	91.2	9″ Ch. 25 lb.	83.
20″ I 65 lb.	624.0	8″ I 18 lb.	75.7	9″ Ch. 13¼ lb.	56.
18″ I 70 lb.	546.1	7″ I 20 lb.	64.5	8″ Ch. 21¼ lb.	63.
18″ I 55 lb.	471.6	7″ I 15 lb.	55.5	8″ Ch. 11¼ lb.	43.
15″ I 100 lb.	640.6	6″ I 17¼ lb.	46.4	7″ Ch. 19¾ lb.	50.
15″ I 80 lb.	565.9	6″ I 12¼ lb.	38.9	7″ Ch. 9¾ lb.	32.
15″ I 75 lb.	491.7	5″ I 14¾ lb.	32.5	6″ Ch. 15½ lb.	34.
15″ I 60 lb.	433.0	5″ I 9¾ lb.	25.6	6″ Ch. 8 lb.	22.
15″ I 55 lb.	363.2	4″ I 10½ lb.	19.2	5″ Ch. 11½ lb.	22.
15″ I 42 lb.	314.2	4″ I 7½ lb.	16.0	5″ Ch. 6½ lb.	16.
12″ I 55 lb.	285.3	3″ I 7½ lb.	10.1	4″ Ch. 7¼ lb.	12.
12″ I 40 lb.	238.9	3″ I 5½ lb.	9.1	4″ Ch. 5¼ lb.	10.
12″ I 35 lb.	202.7	15″ Ch. 55 lb.	306.1	3″ Ch. 6 lb.	7.
12″ I 31½ lb.	192.0	15″ Ch. 33 lb.	222.4	3″ Ch. 4 lb.	5

NOTE—It is preferable to use the lighter or standard weight of the several bracketed pairs. For intermediate weights interpolate to find the value of Q.

TABLE III.

Capacity of Bethlehem I Beams.

Extreme fiber stress 16.000 lbs. per sq. in.

Size.	Q.	Size.	Q.	Size.	Q.
30″ I 120 lb.	1862.9	20″ I 59 lb.	625.1	15″ I 38 lb.	314.
28″ I 105 lb.	1529.1	18″ I 59 lb.	523.2	12″ I 36 lb.	239.
26″ I 90 lb.	1221.3	18″ I 54 lb.	499.2	12″ I 32 lb.	203.
24″ I 84 lb.	1058.7	18″ I 52 lb.	489.1	12″ I 28½ lb.	192.
24″ I 83 lb.	995.7	18″ I 48½ lb.	473.1	10″ I 28½ lb.	143.
24″ I 73 lb.	929.6	15″ I 71 lb.	566.4	10″ I 23½ lb.	131.
20″ I 82 lb.	832.0	15″ I 64 lb.	472.5	9″ I 24 lb.	109.
20″ I 72 lb.	782.4	15″ I 54 lb.	433.6	9″ I 20 lb.	100.
20″ I 69 lb.	676.8	15″ I 46 lb.	344.5	8″ I 19½ lb.	80.
20″ I 64 lb.	651.7	15″ I 41 lb.	324.8	8″ I 17½ lb.	76.

TABLE IV.

Capacity of Bethlehem Girder Beams.

Extreme fiber stress 16,000 lbs. per sq. in.

Size.	Q.	Size.	Q.	Size.	Q.
30″ 200 lb.	3253.3	24″ 120 lb.	1603.2	12″ 70 lb.	478.9
30″ 180 lb.	2913.6	20″ 140 lb.	1565.3	12″ 55 lb.	384.0
28″ 180 lb.	2767.5	20″ 112 lb.	1249.1	10″ 44 lb.	260.3
28″ 165 lb.	2500.3	18″ 92 lb.	942.9	9″ 38 lb.	202.7
26″ 160 lb.	2306.1	15″ 140 lb.	1132.8	8″ 32½ lb.	152.5
26″ 150 lb.	2114.7	15″ 104 lb.	867.7		
24″ 140 lb.	1867.2	15″ 73 lb.	628.3		

TABLE V.

CAPACITY OF ANGLES IN BENDING.

Long Leg Vertical, for Unequal Leg Angles.

Extreme fiber stress 16,000 lbs. per sq. in.

Size.	Q.	Size.	Q.	Size.	Q.
8 x8 x1	84.3	2½x2½x ½	3.9	5 x3 x ⅝	18.9
8 x8 x ½	44.6	2½x2½x ¼	2.1	5 x3 x 5/16	10.1
6 x6 x ¾	35.5	2 x2 x ⅜	1.9	4 x3 x ⅝	12.3
6 x6 x ⅜	18.8	2 x2 x 3/16	1.0	4 x3 x 5/16	6.6
5 x5 x ¾	24.2	7 x3½x ¾	43.8	3½x3 x ⅝	9.4
5 x5 x ⅜	12.9	7 x3½x 7/16	26.7	3½x3 x 5/16	5.1
4 x4 x ¾	15.0	6 x4 x ¾	33.3	3½x2½x ½	7.5
4 x4 x 5/16	6.9	6 x4 x ⅜	17.7	3½x2½x ¼	4.0
3½x3½x ⅝	9.7	6 x3½x ¾	32.5	3 x2½x ½	5.6
3½x3½x 5/16	5.2	6 x3½x ⅜	17.3	3 x2½x ¼	3.0
3 x3 x ½	5.7	5 x3½x ¾	22.8	2½x2 x ⅜	2.9
3 x3 x ¼	3.1	5 x3½x 5/16	10.3	2½x2 x 3/16	1.6

NOTE—Interpolate for intermediate thicknesses.

TABLE VI.

Capacity of Zee-Bars in Bending.

Extreme fiber stress 16,000 lbs. per sq. in.

Size.	Q.	Size.	Q.	Size.	Q.
6 x3½x ⅜	45.0	5 x3¼x ½	41.0	4 x3 1/16x ⅝	32.3
7/16	52.4	9/16	46.0	11/16	35.5
½	59.9	⅝	51.0	¾	38.7
6 x3½x 9/16	61.4	5 x3¼x 11/16	50.5	3 x2 11/16x ¼	10.2
⅝	68.4	¾	55.2	5/16	12.7
11/16	75.2	13/16	59.7	3 x2 11/16x ⅜	13.7
6 x3½x ¾	74.9	4 x3 1/16x ¼	16.8	7/16	15.9
13/16	81.2	5/16	20.9	3 x2 11/16x ½	16.3
⅞	87.5	⅜	24.9	9/16	18.3
5 x3¼x 5/16	28.5	4 x3 1/16x 7/16	25.8		
⅜	34.1	½	29.3		
7/16	39.7	9/16	33.0		

NOTE — Web, flange and thickness increase by same amount in each group.

TABLE VII.

Capacity of Carnegie Tee-Bars in Bending.

Extreme fiber stress 16,000 lbs. per sq. in.

Size, Flange by Stem by Wt. per Ft.	Q.	Size, Flange by Stem by Wt. per Ft.	Q.	Size, Flange by Stem by Wt. per Ft.	Q.
5 x3 x13.6	6.3	3½x4 x10.0	8.3	2½x3 x 7.2	4.6
5 x2½x11.0	4.6	3½x3½x11.9	8.1	2½x3 x 6.2	4.1
4½x3½x15.9	11.4	3½x3½x 9.3	6.4	2½x2¾x 6.8	3.9
4½x3 x 8.6	4.3	3½x3 x11.0	6.0	2½x2¾x 5.9	3.2
4½x3 x10.0	5.0	3½x3 x 8.7	4.7	2½x2½x 6.5	3.1
4½x2½x 8.0	3.0	3½x3 x 7.7	3.8	2½x2½x 5.6	2.7
4½x2½x 9.3	3.5	3 x4 x11.9	10.3	2½x1¼x 3.0	0.5
4 x5 x15.7	16.5	3 x4 x10.6	9.5	2¼x2¼x 5.0	2.2
4 x5 x12.3	13.0	3 x4 x 9.3	8.4	2¼x2¼x 4.2	1.7
4 x4½x14.8	13.6	3 x3½x11.0	7.9	2 x2 x 4.4	1.8
4 x4½x11.6	10.6	3 x3½x 9.8	7.3	2 x2 x 3.7	1.3
4 x4 x13.9	10.8	3 x3½x 8.6	6.5	2 x1½x 3.2	0.8
4 x4 x10.9	8.7	3 x3 x10.1	5.9	1¾x1¾x 3.2	1.0
4 x3 x 9.3	4.7	3 x3 x 9.0	5.4	1½x1½x 2.6	0.7
4 x2½x 8.7	3.3	3 x3 x 7.9	4.6	1½x1½x 2.0	0.6
4 x2½x 7.4	2.9	3 x3 x 6.8	3.9	1¼x1¼x 2.1	0.5
4 x2 x 7.9	2.1	3 x2½x 7.2	3.2	1¼x1¼x 1.7	0.4
4 x2 x 6.7	1.8	3 x2½x 6.2	2.8	1 x1 x 1.3	0.3
3½x4 x12.8	10.6	2¾x2 x 7.4	4.0	1 x1 x 1.0	0.2

Reinforced Concrete Beams.

In the author's book "Concrete" a simple theoretic treatment of reinforced concrete beams is given; also certain rules are derived for the design of such beams. The reader is referred to that book for a discussion of the theory; the rules will be summarized here and a brief outline of the theory given.

The generally accepted standard of reinforced concrete design in America is a hodge-podge of so-called practical mens' patented ideas, given a semblance of authority by eminent investigators and authors, who discuss designs and tests with little or no logical analysis of the stresses in the reinforcement. Sharp bends are made in rods, loose stirrups are assigned stresses that they could not possibly take, steel rods are crowded into the stem of T-beams with no regard to the ability of the concrete to transmit stress into the steel—these and many other absurdities stamp present day practice in reinforced concrete as being on a far lower plane than highway bridge design of 20 years ago. The light highway bridges of the early days of steel bridge are gradually being condemned or failing, not because of wear but because of original weakness; many large reinforced concrete buildings have already failed, at the time when they were nearly completed, because of weak design.

Reinforced concrete is a most excellent form of construction, when properly designed, but American standard design, as exhibited in nearly all the books and in the greater part of the work as illustrated in engineering periodicals, is far from being on a sound basis.

In a paper entitled, "Some Mooted Questions in Reinforced Concrete Design," by the author, read before the American Society of Civil Engineers in March, 1910, common practice in reinforced concrete design is severely criticised in sixteen indictments covering as many phases of that practice. The wide publicity given this paper, (it was reprinted in Engineering News and very fully reviewed in Concrete Engineering), puts it beyond peradventure that

all of the authors and investigators whose methods of design and analysis were attacked in that paper are aware of the attack. Very few have made any defense of any sort. The criticism which followed the reading of the paper, by its illogical analysis, dogmatic assertions and dodging arguments, as well as the strong support given by many eminent engineers, has served to strengthen the stand taken by the author; it proves the crying need of reform in reinforced concrete design.

The foregoing is deemed to have proper place in this book because the book is designed for a class of men who have to deal with buildings, and because it is in building work that the greatest faults in design are exhibited; it is in building work too that the greatest wrecks have occurred.

The author's castigation of common practice, both in reinforced concrete and in steel design, (See "Engineering News," April 11, 1907) has no other motive than a desire to do what he can to place structural design of all kinds on a sound engineering basis.

Following are a number of rules of design for beams in reinforced concrete.

Rule 1. Use no loose stirrups. They interfere with the pouring of the concrete; they cannot possibly take any kind of stress commensurate with their size; they are practically useless until failure has begun in the beam and are therefore illogical as an element of design. Figs. 4 and 5 show how beams with stirrups may fail. The upper loose ends may readily pull out of the concrete.

Fig. 4.

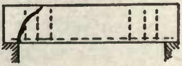


Fig. 5.

Rule 2. Make no sharp bends in reinforcing rods where any considerable stress in the rods exists. At the bend there is set up a side stress in the concrete which the latter is unable to resist. Rods should be given gentle curves, preferably with a radius equal to 20 times the diameter of the rod.

Rule 3. Place no dependence upon hooks or sharp bends in rods as anchors. Anchorage of steel rods may be effected by embedment in concrete to a depth of 50 times the diameter of the rod beyond the point where the full strength of the rod is needed; or it may be effected in a round rod by use of an end nut and a washer or bearing plate, the latter having a bearing surface about twenty times the area of the rod.

Rule 4. Reinforcing rods at the bottom of a reinforced concrete beam extending straight from end to end of span, should have a diameter not more than 1-200 of the length of span.

Rule 5. Reinforcing rods, when curved up to the top of a beam, should run to the end of span and be anchored over the support or run beyond the support so as to take a hold in the concrete. The practice of bending up rods with a sarp bend and of ending them short of the support, or even at the support, and not anchoring them over or beyond the support, is a poor and illogical one.

Rule 6. In beams having a depth of about one-tenth of the span or more, shear reinforcement is needed. Some of the reinforcing rods should be curved up as shown in Figs. 6, 7, 8, 9.

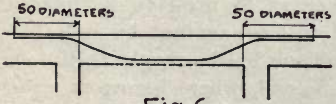


Fig. 6.

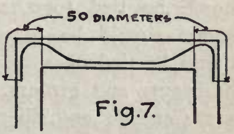


Fig. 7.

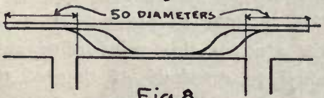


Fig. 8.

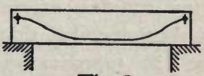


Fig. 9

Note that 50 diameters of the rod is allowed beyond the edge of support for anchorage in Fig. 6, 7, and 8. In Fig. 9 washer plates and nuts are used. If the width of a beam resting on a wall is sufficient, the rod may be curved down, as in Fig. 7, and receive sufficient anchorage within the confines of the beam. The curve should not be sharp but with a radius of about 20 times the diameter of the rod. In a continuous line of beams the portion of the rod that extends into the next beam for anchorage may perform the additional service of acting as upper reinforcement in that beam. Continuity of beams will of course give rise to tension over the suports at the top of the beams. Some of the rods may be bent as shown in Fig. 8 (all of these should run into the next beam for full anchorage), but there is scarcely any need of this; they could all be bent as in Fig. 6, as any local irregularities in the shear can readily be taken care of in the concrete.

Rule 7. The width of a beam should be about equal to the sum of the perimeters of all reinforcing rods that lie near the bottom of the beam from end to end of span. This rule would make the spacing of square rods four times their diameters and of round rods 3.14 times their diameters, with a side distance of 2 and 1.57 diameters respectively on each side of the outer rods. Several tiers of rods in the bottom of a beam should, in general, be avoided.

In addition to the foregoing rules for the design of individual beams it is important to add these two precautions: First, in the general design the entire structure should be tied together so as to preserve its integrity. Beams should be joined to one another by rods; they should be tied into the columns; slabs should be tied into the beams and girders. Second, where beams or girders frame end to end, there should be reinforcing rods near the top running across the support to prevent cracking. It is recommended that the area of this reinforcing steel across the supports be equal to one-half of that of the reinforcement of the beams at middle of span, also that

the rods reach from quarter-span to quarter-span of the beams.

The following unit stresses are recommended:

Tension on steel, 14,000 lbs. per sq. in.

Compression on extereme fiber of the concrete, 600 lbs. per sq. in.

Shear on gross section of concrete about 30 lbs. per sq. in.

Tables VIII to XIII give reinforcement and sizes of rectangle of concrete, as well as a coefficient to determine the carrying capacity, of beams designed according to the foregoing rules and with the unit stresses just given.

In all of these tables

b is the width of concrete beam;

d is the depth out to out of the concrete beam.

The center of the reinforcing rods, at middle of span, is one-eighth of the depth *d* from the bottom of the beam. This makes the depth of concrete protecting and gripping the steel proportional to the magnitude of the rod, as it should be.

The neutral axis of the beam is in all cases assumed to be at the middle of the depth of the concrete beam.

Tables VIII and IX are for a single straight rod. (Of course several rods may be used by making the width in multiples of *b*.) In these tables as in the others, the steel area is 1.07 per cent. This governs the area of beam or the product of *b* and *d*. The minimum value of *b* is the perimeter of a rod. The minimum value of the span length is governed first by 200 times the diameter of the rod and second by twelve times the depth. The first is in accordance with Rule 4; the second is to keep the beam well within Rule 6, since it has no shear reinforcement.

Tables X and XI are for reinforcement with three rods, two of which are straight, and the third is curved up as illustrated in Figs. 6, 7, and 9. The last mentioned rod carries the shear which the concrete is not capable of taking. This rod, being curved up in an approximate parabolic shape, will take the shear incident to its own stress,

or one-third of the total. The remainder of the shear is carried by the concrete. This condition governs the minimum span length. Another governing condition in the minimum span length is 200 times the diameter of rods. It is seen that the width of beam is nowhere less than the sum of the perimeters of the two straight rods. It will also be seen that the depths all lie between 23 and 36 times the diameter of the rods. This, however, has no special significance.

Tables XI and XII are for reinforcement with four rods, two of which are straight and the other two are curved up as illustrated in Figs. 6, 7 and 9. The two curved rods carry one-half the shear and the concrete carries the other half. This condition governs the minimum span length, which is further limited by 200 times the diameter of rods. The width of the beam is nowhere less than 2½ times the perimeter of one rod. The depths all lie between 29 and 38 diameters.

Examples.

(1) Given a 9-in. wall spanning a 6-ft. opening, 5 ft. of wall above the opening. Required a reinforced concrete lintel to carry the wall and 1,000 lbs. per ft. of floor load. The weight of the wall is $90\times5\times6=2{,}700$ lbs., and the floor load is 6,000 lbs. $C=8{,}700\times6.5=56{,}600$. (Note that 6.5 ft. is used as the span to allow for bearing on the wall. By reference to Table VIII it is seen that 4 beams 2¼″ wide and 10½″ deep, with four ½″ square rods for reinforcement, would have a value $C=69{,}400$. This is more than necessary. The lintel would, of course, be 9″ wide and 10½″ deep with four ½″ square rods lying near the bottom. It is assumed that the depth of lintel is included in the height of wall, so that no extra allowance was made for the weight of the lintel. The lintel should rest on the wall for about 10″ at each end. The rods would be about 7½ feet long.

(2) It is desired to design a ribbed floor filled with 12″ tile, the reinforced concrete ribs being about 4″ wide. Span, 16 ft. Over the ribs will be laid wooden sleepers,

filled in between with cinder concrete; on the sleepers will be nailed a 1″ maple floor. Live load 66 lbs. per sq. ft. Each rib supports 16″ of floor. The weights per foot are: Live load, 88; tile, 44; rib (estimated), 50; cinder fill and sleepers, 30; flooring, 5. Total, 217 lbs. per ft. Total load on one rib=217×16=3,472 lbs. C=3,472×16=55,550. By reference to Table VIII it is seen that a 3¾″×14″ rib with one ¾″ square rod for reinforcement has a value C=52,100, which is sufficiently close to the requirements.

(3) Required a reinforced concrete beam carrying a floor load of 800 lbs. per ft. on a clear span of 16 ft. The assumed weight of the beam is 180 lbs. per ft. Total load on beam 980×16=17,480 lbs. C=17,480×16=279,700. Applying tables VIII to XIII inclusive we find the following:

Table VIII. A single reinforcing rod, without end anchorage, will not suffice, since beams with a value of C=279,700 or more have too great a minimum span length. The same is true if we take C=139,900 and use two rods. At C=93,200, using three rods, we could use a beam 16½″ wide and 16½″ deep with three 1″ square rods as reinforcement (as the minimum span is here 16.5 ft.). This would not be a good beam and would not be economical.

Table IX. Neither one nor two rod beams can be used here for the same reason as stated in the foregoing paragraph. It is also seen that when C=93,200 the minimum span is over 18 ft. The conclusion is that a beam for this case needs shear reinforcement.

Table X. Here a beam 10½″ wide and 20½″ deep with three ⅞″ square rods has a value C=311,000, which is more than required. The area of steel reinforcement could be reduced by taking 280/311 of the total and making the two straight rods of smaller section, but as this gives 13/16″, an odd size, for their diameter it is hardly worth while. One of these rods, the middle one, must be curved up and run beyond the edge of support 50 diameters, or otherwise anchored at the supports of the beam.

Table XI. Here we find that a beam 7″ wide and 24″ deep with three ⅞″ round rods comes near meeting all the requirements. One of these rods must be curved up and anchored.

Table XII. In this table the beam 7¾″ wide and 22¾″ deep with four 11/16″ square rods meets all the requirements. Two of these rods must be curved up and anchored.

Table XIII. In this table the beam 7″ wide and 23½″ deep with four ¾″ round rods comes close to meeting all requirements. Two of these rods must be curved up and anchored.

The proper beam for this case may be determined by the desired depth or by the availability of round or square rods.

In the very deep beams one-eighth of the depth of beam may be more than necessary below the center of the rods. The standard beam of these tables has an effective depth of 17/24 of the outside depth of the concrete rectangle. If the rods are dropped so that this effective depth (distance from centroid of compression in the concrete to the center of steel) is increased, the coefficient C of the strength of the beam is increased proportionally. Thus at a depth of 48″ the standard beam would have the rods ⅛ of the depth or 6″ from the bottom. The effective depth is 17/24 of this or 34″. If it is desired to place the rods 4″ above the bottom of the beam, the effective depth is increased by 2″, and C is 36/34 of the tabular value.

TABLE VIII.

Reinforced Concrete Beams with Straight Rods Not Anchored.

R'f'n't One Square Rod Diam. in In.	Sec. of Concrete Beam b in In.	Sec. of Concrete Beam d in In.	Min Sp'n in Ft.	C Prod. of Span in Ft. and Unif'm Load in Pounds.
¼	1	6	6	2480
	1¼	4¾	4¾	1960
	1½	4	4	1650
5/16	1¼	7¼	7¼	4660
	1½	6	6	3830
	1¾	5¼	5¼	3390
⅜	1½	8¾	8¾	8130
	1¾	7½	7½	6970
	2	6½	6½	5990
7/16	1¾	10¼	10¼	12970
	2	9	9	11390
	2¼	8	8	10120
	2½	7¼	7¼	9170
½	2	11¾	11¾	19420
	2¼	10½	10½	17350
	2½	9½	9½	15700
	2¾	8½	8½	14050
9/16	2¼	13¼	13¼	27700
	2½	11¾	11¾	24400
	2¾	10¾	10¾	22500
	3	9¾	9¾	20200
	3¼	9¼	9¼	19300
⅝	2½	14½	14½	37200
	2¾	13¼	13¼	34200
	3	12¼	12¼	31600
	3¼	11¼	11¼	29000
	3½	10½	10½	27100
11/16	2¾	16	16	49900
	3	14¾	14¾	46100
	3¼	13½	13½	41900
	3½	12½	12½	38700
	3¾	11½	11½	35100
¾	3	17½	17½	65100
	3¼	16¼	16¼	60400
	3½	15	15	55800
	3¾	14	14	52100
	4	13¼	13¼	49300
	4¼	12½	12½	46500
⅞	3½	20½	20½	103700
	3¾	19	19	95900
	4	18	18	91100
	4¼	16¾	16¾	84500
	4½	16	16	81000
	4¾	15	15	75700
	5	14½	14½	73400
1	4	23¼	23¼	153200
	4¼	22	22	145400
	4½	20¾	20¾	137200
	4¾	19¾	19¾	130500
	5	18¾	18¾	123900
	5¼	17¾	17¾	117200
	5½	16½	16½	106100
1⅛	4½	26¼	26¼	219600
	4¾	24¾	24¾	206100
	5	23½	23½	195600
	5¼	22½	22½	188200
	5½	21½	21½	179900
	5¾	20½	20½	171200
	6	19¾	19¾	165200
	6¼	18¾	18¾	155600
1¼	5	29¼	29¼	302200
	5¼	27¾	27¾	286400
	5½	26½	26½	273600
	5¾	25¼	25¼	259700
	6	24¼	24¼	249900
	6¼	23¼	23¼	239300
	6½	22½	22½	232400
	6¾	21½	21½	221000
	7	20¾	20¾	213500
1⅜	5½	32	32	399000
	5¾	30¾	30¾	384300
	6	29½	29½	368700
	6¼	28¼	28¼	353000
	6½	27¼	27¼	340500
	6¾	26¼	26¼	328000
	7	25¼	25¼	315500
	7¼	24¼	24¼	302000
	7½	23½	23½	293400
	7¾	22¾	22¾	284100
1½	6	35	35	520600
	6¼	33½	33½	496800
	6½	32¼	32¼	478800
	6¾	31	31	459500
	7	30	30	446200
	7¼	29	29	431400
	7½	28	28	416500
	7¾	27	27	400200
	8	26¼	26¼	390500
	8¼	25½	25½	379300
	8½	24¾	24¾	368200

TABLE IX.

Reinforced Concrete Beams with Straight Rods Not Anchored.

R'f'n't One Round Rod Diam. in In.	Sec. of Concrete Beam b in In.	Sec. of Concrete Beam d in In.	Min Sp'n in Ft.	C Prod. of Span in Ft. and Unif'm Load in Pounds.
¼	¾	6	6	1910
	1	4½	4½	1430
	1¼	4	4	1300
5/16	1	7¼	7¼	3680
	1¼	5¾	5¾	2920
	1½	5	5	2540
⅜	1¼	8¼	8¼	6020
	1½	7	7	5110
	1¾	6¼	6¼	4560
7/16	1¼	11¼	11¼	11200
	1½	9¼	9¼	9090
	1¾	8	8	7930
	2	7	7	6940
½	1½	12¼	12¼	15800
	1¾	10½	10½	13600
	2	9¼	9¼	12000
	2¼	8¼	8¼	10700
9/16	1¾	13¼	13¼	21800
	2	11½	11½	18700
	2¼	10¼	10¼	16700
	2½	9¼	9¼	15200
⅝	2	14½	14½	29400
	2¼	12¾	12¾	25900
	2½	11½	11½	23300
	2¾	10½	10½	21300
11/16	2¼	15½	15½	38000
	2½	13¾	13¾	33500
	2¾	12½	12½	30400
	3	11½	11½	28100
¾	2¼	18¼	18¼	53100
	2½	16½	16½	48200
	2¾	15	15	43800
	3	13¾	13¾	40200
	3¼	12¾	12¾	37300
⅞	2¾	20½	20½	81500
	3	18¾	18¾	74700
	3¼	17¼	17¼	68500
	3½	16	16	63500
	3¾	15	15	59600

R'f'n't One Round Rod Diam. in In.	Sec. of Concrete Beam b in In.	Sec. of Concrete Beam d in In.	Min Sp'n in Ft.	C Prod. of Span in Ft. and Unif'm Load in Pounds.
1	3¼	22½	22½	116500
	3½	21	21	109000
	3¾	19½	19½	101000
	4	18¼	18¼	94400
	4¼	17¼	17¼	89600
	4½	16¼	16¼	84200
1⅛	3½	26½	26½	174100
	3¾	24¾	24¾	162600
	4	23¼	23¼	152800
	4¼	21¾	21¾	142400
	4½	20½	20½	133900
	4¾	19½	19½	127900
	5	18½	18½	121200
1¼	4	28¾	28¾	233300
	4¼	27	27	219100
	4½	25½	25½	206900
	4¾	24	24	193800
	5	23	23	186600
	5¼	21¾	21¾	175900
	5½	20¾	20¾	167700
1⅜	4¼	32½	32½	318000
	4½	30¾	30¾	301400
	4¾	29¼	29¼	287100
	5	27¾	27¾	272400
	5¼	26½	26½	260100
	5½	25¼	25¼	247900
	5¾	24	24	234600
	6	23	23	224800
1½	4¾	34¾	34¾	406000
	5	33	33	385500
	5¼	31½	31½	368000
	5½	30	30	350500
	5¾	28¾	28¾	335900
	6	27½	27½	321300
	6¼	26½	26½	309600
	6½	25½	25½	297700

TABLE X.

Reinforced Concrete Beams--One Rod Curved Up and Anchored.

R'f'n't Three Square Rods Diam. in In.	Sec. of Concrete Beam b in In.	d in In.	Min Sp'n in Ft.	C Prod. of Span in Ft. and Unif'm Load in Pounds.
	2	8¾	6.2	10900
¼	2½	7	5.0	8680
	3	5¾	4.1	7030
	2½	11	7.8	21300
5/16	3	9¼	6.6	17900
	3½	7¾	5.5	14900
	3	13	9.2	35900
⅜	3½	11¼	8.0	31400
	4	10	7.1	27900
	4½	8¾	.6.2	24400
	3½	15¼	10.8	57700
	4	13½	9.6	51300
7/16	4½	12	8.5	45600
	5	10¾	7.6	40800
	4	17½	12.4	86800
	4½	15½	11.0	76600
½	5	14	9.9	69400
	5½	12¾	9.0	63200
	6	11¾	8.3	58300
	4½	19¾	14.0	123900
	5	17¾	12.6	111400
9/16	5½	16	11.3	99700
	6	14¾	10.5	92500
	6½	13½	9.6	83900
	5	22	15.6	170400
	5½	20	14.2	154900
⅝	6	18¼	12.9	141600
	6½	16¾	11.9	129200
	7	15¾	11.2	122000
	7½	14½	10.3	111700
	5½	24	17.0	224400
	6	22	15.6	205700
11/16	6½	20¼	14.3	188800
	7	19	13.5	178100
	7½	17¾	12.6	166400
	8	16½	11.7	154300
	6	26¼	18.6	292900
	6½	24¼	17.2	270500
	7	22½	15.9	251000
¾	7½	21	14.9	234300
	8	19¾	14.0	220300
	8½	18½	13.1	206100
	9	17½	12.4	195200
	7	30¾	21.8	467000
	7½	28½	20.2	432000
⅞	8	26¾	19.0	406000
	8½	25¼	17.9	383000
	9	23¾	16.8	360000
	9½	22½	15.9	341000
	10	21½	15.2	327000
	10½	20½	14.5	311000
	8	35	24.8	694000
	8½	33	23.4	654000
	9	31	22.0	613000
	9½	29½	20.9	585000
1	10	28	19.8	555000
	10½	26¾	19.0	531000
	11	25½	18.1	506000
	11½	24¼	17.2	479000
	12	23¼	16.5	460000
	9	39¼	27.8	982000
	9½	37¼	26.4	934000
	10	35½	25.1	891000
	10½	33¾	23.9	847000
1⅛	11	32¼	22.8	810000
	11½	30¾	21.8	770000
	12	29½	20.9	740000
	12½	28¼	20.0	707000
	13	27¼	19.3	684000
	10	43¾	31.0	1356000
	10½	41¾	29.6	1294000
	11	39¾	28.2	1231000
	11½	38	26.9	1176000
1¼	12	36½	25.9	1131000
	12½	35	24.8	1085000
	13	33¾	23.9	1046000
	13½	32½	23.0	1007000
	14	31¼	22.1	969000
	14½	30¼	21.4	937000
	11	48	34.0	1795000
	11½	46	32.6	1724000
	12	44	31.2	1646000
	12½	42¼	29.9	1581000
	13	40¾	28.9	1528000
1⅜	13½	39¼	27.8	1472000
	14	37¾	26.7	1413000
	14½	36½	25.9	1368000
	15	35¼	25.0	1320000
	15½	34¼	24.3	1284000
	16	33	23.4	1234000
	12	52½	37.2	2343000
	12½	50½	35.8	2254000
	13	48½	34.3	2164000
	13½	46¾	33.1	2086000
	14	45	31.9	2008000
1½	14½	43½	30.8	1941000
	15	42	29.7	1874000
	15½	40½	28.7	1801000
	16	39½	28.0	1762000
	16½	38¼	27.1	1707000
	17	37	26.2	1648000
	17½	36	25.5	1606000

TABLE XI.

Reinforced Concrete Beams--One Rod Curved Up and Anchored.

R'f'n't Three Round Rods Diam. in In.	Sec. of Concrete Beam b in In.	Sec. of Concrete Beam d in In.	Min Sp'n in Ft.	C Prod. of Span in Ft. and Unif'm Load in Pounds.
¼	1½	9¼	6.6	9010
	2	7	5.0	6820
	2½	5½	3.9	5360
5/16	2	10¾	7.6	16400
	2½	8½	6.0	12800
	3	7¼	5.1	11000
⅜	2½	12¼	8.7	26600
	3	10¼	7.3	22300
	3½	8¾	6.2	19000
7/16	3	14	9.9	41700
	3½	12	8.5	35700
	4	10½	7.4	31200
½	3½	15¾	11.2	61300
	4	13¾	9.7	53500
	4½	12¼	8.7	47700
9/16	3½	20	14.2	98600
	4	17½	12.4	86300
	4½	15½	11.0	76400
	5	14	9.9	69000
⅝	4	21½	15.2	131000
	4½	19	13.5	115000
	5	17¼	12.2	105000
	5½	15½	11.0	93600
11/16	4½	23	16.3	168600
	5	20¾	14.7	152500
	5½	19	13.5	139900
	6	17¼	12.2	126400
	6½	16	11.3	117800
¾	5	24¾	17.5	216900
	5½	22½	15.9	197200
	6	20½	14.5	178600
	6½	19	13.5	166200
	7	17¾	12.6	155500
⅞	5½	30½	21.6	362000
	6	28	19.8	333000
	6½	26	18.4	310000
	7	24	17.0	285600
	7½	22½	15.9	268300
	8	21	14.9	249900
1	6½	33¾	23.9	524000
	7	31½	22.3	491000
	7½	29¼	20.7	455000
	8	27½	19.5	428000
	8½	26	18.4	405000
	9	24½	17.4	382000
	9½	23¼	16.5	362000
1⅛	7	39¾	28.2	783000
	7½	37	26.2	727000
	8	34¾	24.6	684000
	8½	32¾	23.2	646000
	9	31	22.0	611000
	9½	29¼	20.7	576000
	10	27¾	19.7	545000
	10½	26½	18.8	522000
1¼	8	43	30.5	1047000
	8½	40½	28.7	986000
	9	38¼	27.1	931000
	9½	36¼	25.7	882000
	10	34½	24.4	840000
	10½	32¾	23.2	797000
	11	31¼	22.1	761000
	11½	30	21.2	730000
1⅜	9	46¼	32.8	1362000
	9½	43¾	31.0	1288000
	10	41½	29.4	1220000
	10½	39½	28.0	1160000
	11	37¾	26.7	1110000
	11½	36¼	25.7	1068000
	12	34¾	24.6	1023000
	12½	33¼	23.5	979000
	13	32	22.7	942000
1½	10	49½	35.1	1735000
	10½	47	33.3	1643000
	11	45	31.9	1577000
	11½	43	30.5	1506000
	12	41¼	29.2	1446000
	12½	39½	28.0	1381000
	13	38	26.9	1330000
	13½	36¾	26.0	1288000
	14	35¼	25.0	1232000

TABLE XII.

Reinforced Concrete Beams--Two Rods Curved Up and Anchored.

R'f'n't Four Square Rods Diam. in In.	Sec. of Concrete Beam: b in In.	Sec. of Concrete Beam: d in In.	Min Sp'n in Ft.	C Prod. of Span in Ft. and Unif'm Load in Pounds.
¼	2½	9¼	4.9	15200
	3	7¾	4.1	12800
5/16	3¼	11¼	6.0	29100
	3¾	9¾	5.2	25200
⅜	4	13¼	7.0	49300
	4½	11¾	6.3	43700
7/16	4¾	15	8.0	75700
	5¼	13½	7.2	67800
½	5	18¾	10.0	124000
	5½	17	9.0	112400
	6	15½	8.2	102100
9/16	5¾	20½	10.9	171200
	6¼	19	10.1	159000
	6¾	17½	9.3	146400
⅝	6½	22½	12.0	233000
	7	20¾	11.0	213500
	7½	19½	10.4	202000
11/16	7¼	24¼	12.9	302000
	7¾	22¾	12.1	284000
	8¼	21½	11.4	269000
¾	7½	28	14.9	417000
	8	26¼	14.0	391000
	8½	24¾	13.2	368000
	9	23¼	12.4	345000
⅞	9	31¾	16.9	643000
	9½	30	16.0	606000
	10	28½	15.2	575000
	10½	27¼	14.5	552000
1	10	37¼	19.8	983000
	10½	35½	18.9	937000
	11	34	18.1	899000
	11½	32½	17.3	860000
	12	31	16.5	817000
1⅛	11½	41	21.8	1369000
	12	39½	21.0	1322000
	12½	37¾	20.1	1262000
	13	36¼	19.3	1210000
	13½	35	18.6	1171000
1¼	12½	46¾	24.9	1932000
	13	45	23.9	1859000
	13½	43¼	23.0	1787000
	14	41¾	22.2	1725000
	14½	40¼	21.4	1663000
	15	39	20.7	1611000
1⅜	14	50½	26.9	2525000
	14½	48¾	25.9	2438000
	15	47	25.0	2347000
	15½	45½	24.2	2273000
	16	44	23.4	2194000
	16½	42¾	22.7	2136000
1½	15	56	29.8	3332000
	15½	54¼	28.8	3228000
	16	52½	27.9	3124000
	16½	51	27.1	3035000
	17	49½	26.3	2945000
	17½	48	25.5	2856000
	18	46¾	24.9	2782000

TABLE XIII.

Reinforced Concrete Beams--Two Rods Curved Up and Anchored.

R'f'n't Four Round Rods Diam. in In.	Sec. of Concrete Beam b in In.	Sec. of Concrete Beam d in In.	Min Sp'n in Ft.	C Prod. of Span in Ft. and Unif'm Load in Pounds.
¼	2	9¼	4.9	12000
	2½	7¼	3.9	9400
5/16	2½	11½	6.1	23300
	3	9½	5.1	19200
⅜	3	13¾	7.3	40200
	3½	11¾	6.3	34200
7/16	3½	16	8.5	63500
	4	14	7.5	55500
½	4	18¼	9.7	94400
	4½	16¼	8.6	84200
9/16	4½	20½	10.9	134000
	5	18½	9.8	121200
⅝	5	23	12.2	186600
	5½	20¾	11.0	167700
	6	19	10.1	153400
11/16	5½	25¼	13.4	248000
	6	23	12.2	225000
	6½	21¼	11.3	208000
¾	6	27½	14.6	321000
	6½	25½	13.6	298000
	7	23½	12.5	274000
⅞	7	32	17.0	508000
	7½	30	16.0	477000
	8	28	14.9	444000
1	8	36¾	19.5	763000
	8½	34½	18.3	717000
	9	32½	17.3	673000
	9½	30¾	16.3	636000
1⅛	9	41¼	21.9	1084000
	9½	39	20.7	1024000
	10	37	19.7	970000
	10½	35¼	18.7	924000
1¼	10	45¾	24.3	1483000
	10½	43¾	23.3	1420000
	11	41¾	22.2	1355000
	11½	40	21.3	1298000
	12	38¼	20.3	1241000
1⅜	11	50½	26.9	1983000
	11½	48¼	25.7	1896000
	12	46¼	24.6	1816000
	12½	44¼	23.5	1734000
	13	42¾	22.7	1679000
1½	12	55	29.2	2570000
	12½	52¾	28.0	2464000
	13	50¾	27.0	2372000
	13½	49	26.1	2290000
	14	47	25.0	2191000

CHAPTER VII.

Girders.

In building work a girder is usually understood to be a large-sized beam, whether rolled or built, particularly a beam that carries smaller floor beams.

The selection of the size of a rolled beam acting as a girder may, of course, be done in the same manner as in the case of simple beams, if the load is uniformly distributed along the beam. When the load is distributed in equal concentrations at equal intervals, the same method may be used with but small error; that is, the total load carried by the floor area tributary to the girder may be used as a uniformly distributed load. This will need correction only where there is an odd number of panels in the girder, as shown in the next paragraph.

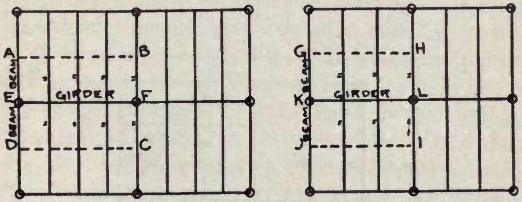


Fig. 1. Fig. 2.

It will be found that the effect on the girder EF, Fig. 1, of the three beam concentrations is the same as the total floor load enclosed by the rectangle ABCD, assumed to be uniformly distributed on the girder. The effect of the two beam concentrations on girder KL, Fig. 2, is less than the load GHIJ, assumed uniformly distributed, by the fraction 1/72. The following rules may then be used in designing girders in such cases.

Rule 1. When there is an even number of equal panels (or an odd number of concentrations), assume the load tributary to a girder (the rectangle ABCD of Fig. 1, the lines AB and CD being midway between girders, etc.), as uniformly distributed on the girder.

Rule 2. When there is an odd number of equal panels (or an even number of concentrations), assume the load tributary to a girder as uniformly distributed on the girder, but deduct 1/72 for 3 panels, 1/200 for 5 panels, 1/392 for 7 panels, etc. The denominator of the fraction is eight times the square of the number of panels. It is seen that the deduction is scarcely worth considering for more than three panels.

In all cases where these rules apply a beam occurs at each column or each end of the girder.

When the girders are not parallel, the method of assuming the load to be uniformly distributed may still be applied with but little error, if the lines AB, DC, etc., be drawn midway between girders.

The general method of finding the bending moment on a girder for any system of concentrated loads is as follows:

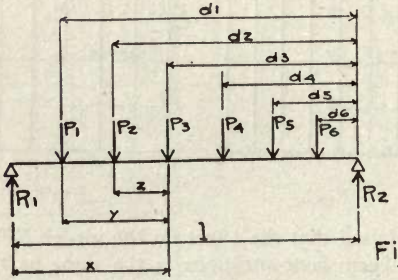


Fig. 3.

$$R_1 = \frac{P_1 d_1 + P_2 d_2 + P_3 d_3 + \text{etc}}{l} \quad \cdots\cdots (1)$$

$$R_2 = (P_1 + P_2 + P_3 + \text{etc.}) - R_1 \quad \cdots\cdots (2)$$

$$M = R_1 x - P_1 y - P_2 z \quad \cdots\cdots (3)$$

M is the bending moment in foot pounds, assuming that all loads are in pounds and all distances are in feet.

The above is on the assumption that the maximum moment occurs under the load P_3. The maximum moment will generally occur under a load near the middle of span. In order to find definitely which load is the critical one, first find the reaction R_1, as indicated, then subtract successively P_1, P_2, P_3, etc., until the load is found where the "shear passes through zero," that is, where a negative value is obtained in this subtracting process.

As a rough check this moment should be nearly equal to half the sum of the products of each several load and the distance to the nearest support. (See Godfrey's Tables, page 43.)

To use this bending moment in the beam and girder tables multiply it by eight and use that product as C in the tables, or divide it by 250 and use that quotient as Q in the tables of rolled beams.

Sometimes an I-beam is reinforced by the addition of top and bottom flange plates. This is not economic construction, but is occasionally necessary to keep down the depth. It is also done sometimes to reinforce existing beams in place. The punching or drilling of holes in the flange of a beam diminishes the strength of that beam in the tension flange, and this must be considered in calculating the reinforcement added by the flange plate. In order to minimize this deduction of area rivet holes should not be located opposite one another in the flanges, but should be alternated, except near the ends of the plate. Here, however, the bending moment is less than at the middle of span.

Table I gives coefficients for finding the load bearing capacity of standard I-beams with flange plates, as well as the length of plate required in terms of the length of span. These tables are figured with two holes out of beam and plate in both top and bottom flanges.

Box girders made of two channels and cover plates or two I-beams and cover plates are frequently used under

TABLE I.

Capacity of I Beams with Flange Plates.

Size of I-Beam	Size of Top and Bottom Flange Plate in Inches.	Length of Fl'nge Plate / Portion of Span.	Q Prod. of Safe Ld. in T'ns & Sp'n in Feet.
10″ 25 lb.	7x⅜	.79	187.1
10″ 25 lb.	7x⅝	.87	254.2
12″ 31.5 lb.	7x⅜	.74	258.3
12″ 31.5 lb.	7x⅝	.83	337.9
15″ 42 lb.	8x½	.77	469.4
15″ 42 lb.	8x¾	.84	588.6
18″ 55 lb.	9x½	.75	693.7
18″ 55 lb.	9x¾	.82	859.8
20″ 65 lb.	9x½	.70	857.6
20″ 65 lb.	9x¾	.79	1041.0
24″ 80 lb.	10x½	.70	1244.0
24″ 80 lb.	10x¾	.77	1495.0

TABLE II.

Capacity of Channel Beams with Cover Plates.

2 Channels Size.	Size of Top and Bottom Cover Plate in Inches.	Length of Bot. Plate / Portion of Span.	Q Prod. of Safe Ld. in Tons & Span in Feet.
7″ 9.75 lb.	9x¼	.83	112.6
7″ 9.75 lb.	9x½	.93	180.6
8″ 11.25 lb.	9x¼	.80	134.5
8″ 11.25 lb.	9x½	.90	209.3
9″ 13.25 lb.	9x¼	.78	161.8
9″ 13.25 lb.	9x½	.89	245.7
10″ 15 lb.	12x⅜	.87	303.4
10″ 15 lb.	12x⅝	.93	436.9
12″ 20.5 lb.	12x½	.86	491.4
12″ 20.5 lb.	12x¾	.91	651.0
15″ 33 lb.	18x½	.86	993.0
15″ 33 lb.	18x¾	.91	1310.0

walls and sometimes in other locations. Table II gives a number of such box girders and coefficients for finding the load bearing capacity. Generally the top plate is run the full length. The bottom plate may be made shorter, as indicated in the table.

The values in the third column of Tables I and II are .06 greater than the theoretical length of cover plate required. This is to allow for rivets near the ends of the plates. The rivets should be spcaed 3 inches apart for a short ditsance at the ends of the plates.

Examples.

(1) Given a 12-inch 31½-lb. I-beam in place that is to be reinforced so that on a span of 16 ft. it will carry 20 tons. The value of Q should be 20×16=320. By reference to Table I it is seen that this comes between the two values of Q for a 12-inch I-beam. By interpolation it is found that 7″×9/16″ flange plates will be required. Interpolating again these plates should be about .81 of the span in length, or 13 feet.

(2) Given a 9-in. wall six feet high carrying a roof slab, whose total load is 900 lbs. per foot. The span is 15 feet. The weight of the wall is 90×6×15=8,100, and the roof load is 900×15=13,500, a total of 10.8 tons. Q is 10.8×15=162. By reference to Table II it is seen that 2 9-in. 13.25-lb. channels with 9″×¼″ top and bottom cover plates will meet the requirements. The top plate may be full length and the bottom plate .78×15, or say 12 feet long.

(3) Given a bay window, the walls of which are supported on columns at the first floor. Find the size of box girder of channels and plates for the following data: Span, 12 ft.; weight of wall, 72,000 lbs.; weight of floors, 48,000 lbs. The total load carried is 120,000 lbs., or 60 tons. Q is 60×12=720. In Table II it is seen that 2 15-in. channels and two 18″×½″ plates would have more strength than necessary. It is also seen that ⅛″ in thickness of the cover plates adds or deducts about 150 in the value of Q. Hence with 18″×⅜″ plates Q is about 840. This size of plates could then be used. Both plates should

be full length; the lower plate can act as a bearing plate on the column.

PLATE GIRDERS.

In a plate girder there are several points of design that must be considered.

First—The section of the flanges must be sufficient to take the longitudinal stresses resulting from the maximum bending moment.

Second—The web plate must be thick enough to take the maximum shear.

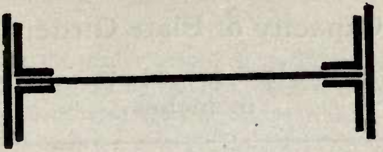
Third—The rivets in the flange angles connecting the same to the web must be sufficient to take the flange stress from web to flange.

Fourth—The web plate must be stiffened against buckling, if it is not of sufficient rigidity in itself to take the shear without buckling.

Fifth—There must be end stiffeners designed to take the full reaction of the girder and to transmit the same into the web plate.

Tables III and IV give 170 girders and coefficients to determine the capacity of the same. The number may be indefinitely extended by interpolating for different thicknesses of flange plates or angles. Also by noting the increase in the value of Q_1 and Q_2 per inch increase in depth of web, in the various pairs of groups of girders having the same flanges, the value of these coefficients for different depths may readily be found. (These tables are based on tables in "Godfrey's Tables," pages 94 to 98 inclusive.) The depth of girder back to back of angles is ¼ in. more than the depth of web plate. The coefficient Q_2 is to be used only when the web plate of the entire girder is in one piece or is spliced for bending with extra flange plates or extra side plates near the top and bottom of the girder to take the flange stress assumed to be carried by one-eighth of the web of the girder. The unit stress of 15,000 lbs. per sq. in. is used here because it is believed that a member made up of several pieces acting together does not have the same uniformity of distribu-

TABLE III.



Capacity of Plate Girders.

Unit stress 15,000 lbs. per sq. in. All Dimensions in inches.

Angles	C'v'r Plate	W'b	Q_1 No Part of W'b Inc. in Flngs.	Q_2 ⅛ of Web Inc. in Flngs.	Angles	C'v'r Plate	W'b	Q_1 No Part of W'b Inc. in Flngs.	Q_2 ⅛ of Web Inc. in Flngs.
2½x2½x¼		18x¼	163.2	213.0	3½x3 x5/16		20x5/16	308	386
2½x2½x½			301.2	351.0	3½x3 x⅝			575	652
2½x2½x¼	6x¼		262.2	312.0	3½x3 x5/16	8x5/16		510	593
2½x2½x¼	6x½		363.0	413.4	3½x3 x5/16	8x⅝		718	796
2½x2½x½	6x½		501.6	551.4	3½x3 x⅝	8x⅝		986	1063
2½x2½x¼		22x¼	201.6	277.2	3½x3 x5/16		26x5/16	407	539
2½x2½x½			373.2	448.2	3½x3 x⅝			763	894
2½x2½x¼	6x¼		322.2	397.2	3½x3 x5/16	8x5/16		668	800
2½x2½x¼	6x½		444.6	520.2	3½x3 x5/16	8x⅝		934	1066
2½x2½x½	6x½		616.2	691.8	3½x3 x⅝	8x⅝		1291	1423
3 x2½x¼		18x¼	184.8	234.6	4 x3 x5/16		20x5/16	340	418
3 x2½x½			345.0	394.8	4 x3 x⅝			636	713
3 x2½x¼	7x¼		306.6	356.4	4 x3 x5/16	9x5/16		574	651
3 x2½x¼	7x½		431.4	481.8	4 x3 x5/16	9x⅝		814	892
3 x2½x½	7x½		592.2	642.6	4 x3 x⅝	9x⅝		1111	1189
3 x2½x¼		22x¼	228.0	303.6	4 x3 x5/16		26x5/16	449	581
3 x2½x½			427.2	502.8	4 x3 x⅝			842	974
3 x2½x¼	7x¼		376.2	451.8	4 x3 x5/16	9x5/16		750	882
3 x2½x¼	7x½		527.4	603.0	4 x3 x5/16	9x⅝		1059	1192
3 x2½x½	7x½		726.6	802.8	4 x3 x⅝	9x⅝		1453	1585
3½x2½x¼		18x¼	207.6	258.0	5 x3 x⅜		22x5/16	527	622
3½x2½x½			389.4	439.2	5 x3 x¾			984	1078
3½x2½x¼	8x¼		352.8	402.6	5 x3 x⅜	11x⅜		921	1016
3½x2½x¼	8x½		501.0	552.0	5 x3 x⅜	11x¾		1327	1422
3½x2½x½	8x½		683.4	733.2	5 x3 x¾	11x¾		1785	1879
3½x2½x¼		22x¼	256.8	332.4	5 x3 x⅜		28x5/16	679	831
3½x2½x½			481.8	556.8	5 x3 x¾			1271	1423
3½x2½x¼	8x¼		432.6	508.2	5 x3 x⅜	11x⅜		1177	1329
3½x2½x¼	8x½		612.6	688.2	5 x3 x⅜	11x¾		1687	1840
3½x2½x½	8x½		837.6	913.2	5 x3 x¾	11x¾		2280	2432
3 x3 x5/16		20x5/16	278.4	355.8	3½x3½x⅜		22x5/16	435	529
3 x3 x⅝			514.8	592.2	3½x3½x¾			805	898
3 x3 x5/16	7x5/16		448.2	526.2	3½x3½x⅜	8x⅜		701	796
3 x3 x5/16	7x⅝		622.8	700.8	3½x3½x⅜	8x¾		977	1070
3 x3 x⅝	7x⅝		860.4	937.8	3½x3½x¾	8x¾		1349	1443
3 x3 x5/16		26x5/16	369.0	500.4	3½x3½x⅜		28x5/16	564	716
3 x3 x⅝			684.0	815.4	3½x3½x¾			1047	1198
3 x3 x5/16	7x5/16		588.0	720.0	3½x3½x⅜	8x⅜		901	1053
3 x3 x5/16	7x⅝		811.8	944.4	3½x3½x⅜	8x¾		1246	1398
3 x3 x⅝	7x⅝		1127.4	1260.0	3½x3½x¾	8x¾		1732	1883

TABLE IV.

Capacity of Plate Girders.

Unit stress 15,000 lbs. per sq. in. All dimensions in inches.

Angles	C'v'r Plate	W'b	Q No Part of W'b Inc. in Flngs.	Q_2 ⅛ of Web Inc. in Flngs.
5 x3½x⅜		24x5/16	613	725
5 x3½x¾			1147	1259
5 x3½x⅜	11x⅜		1042	1154
5 x3½x⅜	11x¾		1483	1595
5 x3½x¾	11x¾		2018	2130
5 x3½x⅜		30x5/16	776	951
5 x3½x¾			1456	1631
5 x3½x⅜	11x⅜		1309	1484
5 x3½x⅜	11x¾		1854	2029
5 x3½x¾	11x¾		2535	2711
6 x3½x7/16		26x5/16	790	921
6 x3½x⅞			1463	1595
6 x3½x7/16	13x7/16		1444	1576
6 x3½x7/16	13x⅞		2121	2253
6 x3½x⅞	13x⅞		2792	2924
6 x3½x7/16		32x5/16	982	1180
6 x3½x⅞			1825	2023
6 x3½x7/16	13x7/16		1784	1983
6 x3½x7/16	13x⅞		2608	2808
6 x3½x⅞	13x⅞		3449	3648
4 x4 x⅜		24x5/16	556	668
4 x4 x¾			1039	1150
4 x4 x⅜	9x⅜		893	1005
4 x4 x⅜	9x¾		1240	1352
4 x4 x¾	9x¾		1724	1836
4 x4 x⅜		30x5/16	707	882
4 x4 x¾			1326	1500
4 x4 x⅜	9x⅜		1126	1301
4 x4 x⅜	9x¾		1555	1729
4 x4 x¾	9x¾		2174	2348
6 x4 x7/16		26x5/16	831	962
6 x4 x⅞			1551	1682
6 x4 x7/16	13x7/16		1485	1617
6 x4 x7/16	13x⅞		2162	2294
6 x4 x⅞	13x⅞		2880	3012
6 x4 x7/16		32x5/16	1036	1234
6 x4 x⅞			1939	2136
6 x4 x7/16	13x7/16		1838	2037
6 x4 x7/16	13x⅞		2662	2862
6 x4 x⅞	13x⅞		3563	3762
6 x6 x½		30x⅜	1277	1486
6 x6 x⅞			2126	2334
6 x6 x½	14x½		2197	2407
6 x6 x½	14x1		3148	3359
6 x6 x⅞	14x1		3994	4204

Angles	C'v'r Plate	W'b	Q No Part of W'b Inc. in Flngs.	Q_2 ⅛ of Web Inc. in Flngs.
6 x6 x½		40x⅜	1753	2122
6 x6 x⅞			2925	3294
6 x6 x½	14x½		2972	3344
6 x6 x½	14x1		4223	4595
6 x6 x⅞	14x1		5393	5765
6 x6 x½		50⅜	2227	2807
6 x6 x⅞			3724	4303
6 x6 x½	14x½		3747	4330
6 x6 x½	14x1		5298	5881
6 x6 x⅞	14x1		6792	7374
6 x6 x½		60x⅜	2702	3539
6 x6 x⅞			4523	5359
6 x6 x½	14x½		4522	5362
6 x6 x½	14x1		6372	7212
6 x6 x⅞	14x1		8190	9030
6 x6 x½		70x⅜	3178	4318
6 x6 x⅞			5322	6462
6 x6 x½	14x½		5297	6438
6 x6 x½	14x1		7445	8592
6 x6 x⅞	14x1		9588	10730
6 x6 x½		80x⅜	3652	5145
6 x6 x⅞			6120	7614
6 x6 x½	14x½		6072	7566
6 x6 x½	14x1		8520	10020
6 x6 x⅞	14x1		10990	12490
6 x6 x½		90x7/16	4127	6330
6 x6 x⅞			6918	9120
6 x6 x½	15x ½		7074	9282
6 x6 x½	15x1½		13070	15280
6 x6 x⅞	15x1½		15850	18070
6 x6 x½		100x7/16	4602	7326
6 x6 x⅞			7716	10440
6 x6 x½	15x ½		7872	10600
6 x6 x½	15x1½		14510	17250
6 x6 x⅞	15x1½		17630	20360
8 x8 x½		100x½	6474	9582
8 x8 x⅞			10970	14090
8 x8 x½	18x ½		10500	13610
8 x8 x½	18x1½		18680	21800
8 x8 x⅞	18x1½		23180	26300
8 x8 x½		120x½	7824	12310
8 x8 x⅞			13270	17750
8 x8 x½	18x ½		12650	17140
8 x8 x½	18x1½		22430	26920
8 x8 x⅞	18x1½		27870	32370

tion of stress that single pieces such as I-beams would show.

By selecting the girder according to Tables III and IV, the first requisite may be fulfilled. When the load is not a uniformly distributed load or its equivalent, the maximum bending moment must be found in ft.-lbs., and by dividing this by 250 Tables III and IV may be used, since 1/250 of the bending moment is equal to the value Q of these tables.

The shear in the web plate should not exceed about 7,500 lbs. per sq. in. of the gross section of the web. Hence to determine whether the second requisite is fulfilled it must be seen whether or not the area of the web agrees with this condition. The maximum shear on a girder in a simple span is the end reaction. In the case of a uniformly loaded beam this is one-half of the total load carried. In other cases, as for concentrated loads, use the methods of equations (1) and (2) to find the reactions. The greater of these is the maximum shear. The gross area of the web is the full section of the plate, no deduction being made for rivets. Thus a 62″×5/16″ web plate will take a shear of 62×5/16×7,500=145,300 pounds.

The spacing of rivets in the flange angles is a detail usually left to the bridge shop, where the girder is made or to the draftsman; but it is also very often carried out in an improper manner. Sometimes the design is such as not to allow rivets enough in the leg of angle connecting to the web. Two rows of rivets may be needed where it is only possible to use one, as when 6″×3½″ angles are used with the 3½-in. leg against the web. The designer must bear this in mind in selecting the section of girder. If the shear is such as to require two rows of rivets, a six-inch angle leg should be used against the web.

Flange plates such as 14″×1″ or 15″1½″ may be made up of two or more plates as 2 14″×½″ plates or 3 15″×½″ plates.

Usually one top flange plate is made nearly or quite the full length of the girder. The theoretical length of the other flange or cover plates may be found by the following formula:

$$\frac{\text{Total flange area}}{\text{Square of span in feet}} = \frac{\text{Area of cover plate}}{\text{Square of length of cover plate}}$$

To this theoretical length of the cover plate add a foot or more.

This formula applies as stated for the outside cover plate. For the second cover plate substitute for "area of cover plate" the area of the first plus the second; for the third cover plate this "area of cover plate" is the area of the first three, etc.

TABLE V.

RIVET PITCH IN FLANGES OF GIRDERS FOR VARIOUS UNIT SHEARS.

On basis of bearing value of rivets at 18,000 lbs. per sq. in.

Unit Shear	8000	7000	6000	5000	4000	3000	2000
⅞" Rivets.....	1.97	2.25	2.63	3.15	3.94	5.25	7.87
¾" Rivets.....	1.69	1.93	2.25	2.70	3.38	4.50	6.75
⅝" Rivets.....	1.41	1.61	1.88	2.25	2.81	3.75	5.62

The rivet spacing in the flange angles for rivets through the web may be found by Table V, by determing first the shear per sq. in. in the web. The closest spacing is required near the ends of span, and near the middle of span the spacing reaches a maximum, which is generally six inches. A few different spaces will be employed, using the closest for a few feet at the ends, then stepping up at intervals to the maximum.

The thickness of the web plate should not be less in any case than about 1/200 of the clear depth between the flange angles, as thin wide plates are apt to have buckles due to cooling, which are very hard to remove.

TABLE VI.

SHEAR ON PLATE GIRDER WEBS.

$\frac{d}{t}$	Allowed Shr. per Sq. In.	$\frac{d}{t}$	Allowed Shr. per Sq. In.	$\frac{d}{t}$	Allowed Shr. per Sq. In.
40	7830	80	3830	140	1590
50	6550	90	3240	160	1260
60	5450	100	2770	180	1020
70	4560	120	2070	200	840

d=either clear depth between flange angles or clear distance between stiffeners.

t=thickness of web.

When the thickness of web plate is relatively less than that shown in Table VI, stiffeners are needed. Thus, suppose a ⅜-in. girder web is 40 in. between flange angles in clear depth and is subject to 5,000 lbs. per sq. ft. of shear. The ratio of depth to thickness is here 107. By Table VI a shear of about 2,500 lbs. per sq. in. is allowed. Stiffeners are needed. At 5,000 lbs. per sq. in. a ratio of depth to thickness of 65 is allowed This would require about 24 in. in the clear between stiffeners. A pair of stiffener angles should then be used about two feet from the end of girder. If the shear at this stiffener is less than at the end of girder, the space to the next stiffener will be more. At the section where the shear is 2,500 lbs. per sq. in. no stiffeners are required. If this were a uniformly loaded girder, no stiffeners would be required in the middle half. One quarter of the girder at each end would need stiffeners varying in clear spacing from 24 in. at the ends of span to 40 inches.

The end stiffeners of a girder have an office to perform which is more than the mere stiffening of the web. They should be designed to take the full end reaction of the girder and transmit it into the web. A unit stress of about 15,000 lbs. per sq. in. may be used in determining the area required. Thus suppose the end reaction of a girder is 240,000 lbs. At 15,000 lbs. per sq. in. this would require 16 sq. in. in the angles. This could be made up

of 4 angles 5x3½x½. These four angles must deliver the load of 240,000 lbs. to the web of the girder. At 18,000 lbs. per sq. in. the bearing value, say of a ⅞-in. rivet in a ½-in. web. is 7880 lbs. Thirty rivets are required, or 15 in each pair of angles.

Examples:

(1) Given a girder of 40 ft. span carrying a load of 3,000 lbs. per ft. The total load on the girder is 60 tons. Q is $40 \times 60 = 2{,}400$. By Table IV a girder having a 30″x⅜″ web, 6″x6″x½″ flange angles and a 14″x½″ cover plate will suffice, if the web plate is in one piece or spliced for bending. The end shear is $3{,}000 \times 20 = 60{,}000$ lbs. On the 30″x⅜″ web this is 5,330 lbs, per sq. in. By Table V the rivet spacing of ¾″ rivets should be about 2.5″ near the ends. By Table VI the clear depth of web may be 60 times the thickness, which is 22.5″; as the clear depth here is 30—12 or 18″, no intermediate stiffeners are needed. The end stiffeners, for the reaction of 60,000 lbs., require $60{,}000 \div 15{,}000$ or 4 sq. in. On account of the 6″ flange angles the stiffener angles should not be less than say 2 angles 5″×3½″×⅜″, which would be more area than necessary. For the length of the cover plate, the total flange area is 1.41 (⅛ of web) +11.50 (angles)+7 (cover plate)=19.91 sq. in.

By the formula

$$\frac{19.91}{40 \times 40} = \frac{7}{\text{Square of length.}}$$

from which the theoretical length is 23.7 ft. The plate would be made 25 feet long.

Other girders could be selected that would be more economical than the one chosen. For example, by interpolation it is seen that a girder with a 32″x5-16″ web 6″x4″x7-16″ angles and a 13″x¾″ cover plate would do. The web of this girder need not be spliced for bending, since Q_1 is used. The shear on the web is 6,000 lbs. per sq. in., and this would require rivet spacing of 2¼″, which is the minimum limit for a single row.

(2) Given a girder of 18 ft. span and having a concentrated load at its center of 10,000 lbs. A concentrated load at the center of a girder is equivalent, so far as bending is concerned, to a uniformly distributed load of double the amount. The equivalent uniform load on this girder is then 20,000 lbs. or 10 tons. Q is $18 \times 10 = 180$. By Table III the girder could have an 18"x¼" web and 3"x2½"x¼" flange angles. The shear is 5,000 lbs. or 1,100 lbs. per sq. in., which is low.

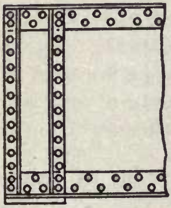


Fig. 4.

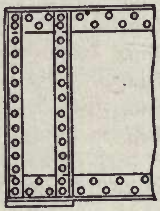


Fig 5.

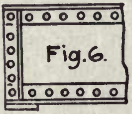


Fig. 6.

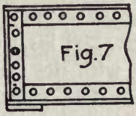


Fig. 7

End stiffeners should be turned as shown in Figs. 4 and 6 and not as in Figs. 5 and 7. In the latter case the outstanding legs of angles, which take the greater part of the bearing, are over the edge of bearing plate and end of angles, whereas they should be well back.

Where there is a heavy concentrated load, such as a column, supported by a girder, the stiffener under the same must be designed to take the column load. The sectional area and the rivets in the web may be found as for end stiffeners, as illustrated above.

The bearing plate of a girder resting on a wall must be designed to give a pressure not to exceed certain limits and must be stiff enough in itself to distribute the load

over this area. The standards given in Table VII will suffice for ordinary cases of rolled beams.

TABLE VII.

CARNEGIE STANDARD WALL PLATES.

Depth of Beam	Size of Plate	Weight
24-in.	16x 1x16	73 lbs.
20-in.	16x 1x16	73 lbs.
18-in.	16x 1x16	73 lbs.
15-in.	12x¾x16	41 lbs.
12-in.	12x¾x12	31 lbs.
10 and 9-in.	8x⅝x12	17 lbs.
8 and 7-in.	8x⅝x 8	12 lbs.
6 and 5-in.	6x½x 6	5 lbs.
4 and 3-in.	6x½x 6	5 lbs.

Smaller dimension is in direction of beam for plates not square.

On cut stone or concrete the pressure allowed per sq. in. on bearing plates may be taken as 300 lbs.; on brick in cement mortar, 200 lbs.; on brick in lime mortar, 110 lbs. These values, with the load, will determine the area of the bearing plate. The length of girder resting on the wall will limit the dimension of the plate in one direction. If the size of plate required necessitates projection of the plate beyond the angles of the girder, this projection should not be too great for the thickness of the plate. In fact the flange angles of the girder are not always sufficient to stiffen the bearing plate to their edge. It may be better in many cases to take the distance out to out of stiffener angles as the stiffened portion of the bearing plate.

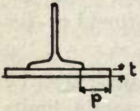


Fig. 8.

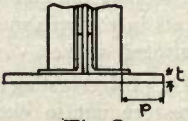


Fig. 9

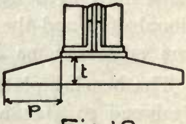


Fig. 10.

In Figs. 8 and 9 the projection p should not exceed seven times t for bearing plates on brick walls in lime mortar; it should not exceed five times t for brick in cement mortar, nor four times t for cut stone or concrete walls.

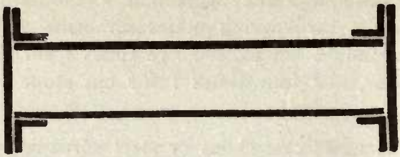
In special cases cast shoes or steel spreading beams are necessary to give sufficient bearing against the wall and to have sufficient stiffness at the same time. Cast-iron shoes, as in Fig. 10, may be made with p equal to twice t.

BOX GIRDERS.

Box girders, composed of two web plates, four flange angles, and cover plates, are often used in buildings, as under walls or suporting heavy loads. Table VIII gives a number of such girders with co-efficients for finding their capacity. These are also figured at 15,000 lbs. per sq. in. on the steel. The depth back to back of angles is ¼ inch greater than the depth of web. The size of rivet assumed is ¾ in.

The spacing of rivets in the flanges of a box girder is not so simply determined as in the case of a plate girder, since single shear on the rivet generally determines its value and not bearing. But as there are two rows of rivets to rely upon, ordinary close spacing will be ample. To find the rivet spacing required for any given shear, divide this shear by the depth of the girder in feet. (In exact work it should be the effective depth or the distance between the centers of gravity of the flanges, but the depth of web is close enough for ordinary work.) Then divide this shear per foot by the single shear value of one rivet. This quotient is the number of rivets required in one foot along the flange. In the case of a box girder these rivets are in two rows. Thus, suppose a 30″ girder has an end shear of 60,000 lbs. The shear per foot is $60{,}000 \div 2.5 = 24{,}000$ lbs. The value of a ¾-in. rivet in single shear at 9,000 lbs. per sq. in. is 3,980 lbs.; $24{,}000 \div 3{,}980 = 6$ rivets per ft. In two rows this would require 4-inch spacing.

TABLE VIII.



Capacity of Box Girders.

Unit stress 15,000 lbs. per sq. in. All Dimensions in inches.

Angles	C'v'r Plate	W'b	Q_1 No Part of W'b Inc. in Flngs.	Q_2 ⅛ of Web Inc. in Flngs.	Angles	C'v'r Plate	W'b	Q_1 No Part of W'b Inc. in Flngs.	Q_2 ⅛ of Web Inc. in Flngs.
3 x3 x5/16	12x⅜	24x5/16	813	1038	3½x3½x½	22x⅜	42x5/16	2743	3431
3 x3 x5/16	12x¾		1299	1527	3½x3½x½	22x¾		4390	5080
3 x3 x⅝	12x¾		1587	1811	3½x3½x¾	22x¾		4870	5560
3 x3 x5/16	12x⅜	30x5/16	1018	1369	3½x3½x½	22x⅜	48x5/16	3139	4041
3 x3 x5/16	12x¾		1620	1974	3½x3½x½	22x¾		5014	5918
4 x3 x⅝	12x¾		1987	2337	3½x3½x¾	22x¾		5567	6470
3½x3½x⅜	12x⅜	30x5/16	1195	1545	4 x4 x⅜	22x⅜	48x5/16	3006	3908
3½x3½x⅜	12x¾		1799	2150	4 x4 x⅜	22x¾		4881	5785
3½x3½x¾	12x¾		2319	2669	4 x4 x¾	22x¾		5902	6804
3½x3½x⅜	12x⅜	36x5/16	1440	1946	4 x4 x⅜	22x⅜	54x5/16	3386	4525
3½x3½x⅜	12x¾		2159	2666	4 x4 x⅜	22x¾		5489	6630
3½x3½x¾	12x¾		2792	3297	4 x4 x¾	22x¾		6644	7782
4 x4 x⅜	12x⅜	36x5/16	1563	2070	3½x3½x½	24x7/16	54x5/16	4122	5261
4 x4 x⅜	12x¾		2284	2791	3½x3½x½	24x⅞		6806	7950
4 x4 x¾	12x¾		3035	3541	3½x3½x¾	24x⅞		7454	8593
4 x4 x⅜	12x⅜	42x5/16	1830	2518	3½x3½x½	24x7/16	60x5/16	4583	5986
4 x4 x⅜	12x¾		2667	3354	3½x3½x½	24x⅞		7558	8967
4 x4 x¾	12x¾		3553	4240	3½x3½x¾	24x⅞		8280	9685
3 x3 x⅜	18x⅜	30x5/16	1437	1788	4 x4 x½	24x7/16	60x5/16	4867	6269
3 x3 x⅜	18x¾		2382	2736	4 x4 x½	24x⅞		7860	9270
3 x3 x⅝	18x¾		2679	3031	4 x4 x¾	24x⅞		8700	10110
3 x3 x⅜	18x⅜	36x5/16	1727	2137	4 x4 x½	24x7/16	66x5/16	5357	7058
3 x3 x⅜	18x¾		2855	3366	4 x4 x½	24x⅞		8640	10350
3 x3 x⅝	18x¾		3214	3723	4 x4 x¾	24x⅞		9570	11280
3½x3½x⅜	18x⅜	36x5/16	1850	2358	6 x6 x½	30x½	70x⅜	8240	10540
3½x3½x⅜	18x¾		2989	3498	6 x6 x½	30x1		13310	15600
3½x3½x¾	18x¾		3620	4126	6 x6 x¾	30x1		14790	17080
3½x3½x⅜	18x⅜	42x5/16	2162	2850	6 x6 x½	30x½	80x⅜	9440	12430
3½x3½x⅜	18x¾		3484	4172	6 x6 x½	30x1		15200	18200
3½x3½x¾	18x¾		4227	4915	6 x6 x¾	30x1		16910	19910
4 x4 x⅜	18x⅜	42x5/16	2308	2996	6 x6 x½	36x½	90x½	11990	17050
4 x4 x⅜	18x¾		3630	4321	6 x6 x½	36x1		19840	24900
4 x4 x¾	18x¾		4516	5205	6 x6 x¾	36x1		21770	26840
4 x4 x⅜	18x⅜	48x5/16	2643	3545	6 x6 x½	36x½	100x½	13330	19580
4 x4 x⅜	18x¾		4148	5052	6 x6 x½	36x1		22040	28290
4 x4 x¾	18x¾		5169	6070	6 x6 x¾	36x1		24190	30440

Examples:

(1) Given a floor girder of 30 ft. span, to be limited in depth to about 24 inches, the total load being 3,600 lbs. per ft. The load in tons is $1.8 \times 30 = 54$. Q is $54 \times 30 = 1620$. A box girder with 24″x5-16″ webs, 3″x3″x⅝″ angles and 12″x¾″ cover plates will suffice. The angles could be 9-16″ thick, if the web plates are in one piece.

(2) Given a box girder on a 60-foot span supporting a 24-inch wall, the total load per ft. being 5,000 lbs. The load carried is $5{,}000 \times 60 = 300{,}000$ lbs. or 150 tons. Q is $150 \times 60 = 9{,}000$. By Table VIII it is seen that a box girder with two 66″x5-16″ webs 4″x4″x⅝″ angles and 24″x⅞″ cover plates would do. The web plates need not be spliced for bending, as Q_1 is used, but of course they should be spliced for shear. The end shear of this girder is 150,000 lbs. On the two 66″x5-16″ webs this is 3,640 lbs. per sq. in. The webs need stiffeners. These should be spaced, according to Table VI, about 83 times the thickness of the web in the clear or 26 inches at the end of girder. At quarter points the spacing of stiffeners is about 40 inches; etc. For flange rivets, the shear per foot at end of span is $150{,}000 \div 5.5 = 27{,}300$ lbs. At 3,980 lbs. per rivet 6.9 rivets are required per ft. or 3.5″ spacing in each of the two rows.

Box girders should have occasional inside diaphragms composed of a plate and angles riveted to the webs These are quite necessary where the load is applied to one side of the girder, so as to insure the uniform distribution of the load into the two sides of the girder. A diaphragm could take the place of a pair of stiffener angles in a deep girder.

Box girders are sometimes used as cantilever girders in foundation work to support wall columns that must have their foundation located back from the center of the column. In such case, to use Table VIII the bending moment in the girder should be found in foot-pounds and this moment divided by 250, which will give an equivalent of Q in the table.

Example of cantilever girder.

Given a cantilever girder supporting a column having a load of 120,000 lbs., the overhang being 5 ft. The bending moment is $120{,}000 \times 5 = 600{,}000$ ft.-lbs. Dividing this by 250, Q is found to be 2,400. By Table VIII, interpolating, it is found that the girder could be composed of 2 webs 42"x5-16", 4 angles 4"x4"x3/8", and 2 plates 12"x5/8".

CHAPTER VIII.

Trusses.

The designing of a truss involves first the calculation of the stresses in the several members and the selection of suitable members to take these stresses. The bending stresses as well as the direct stress must be found for any members subject to transverse loading, and such members must be designed to resist both kinds of stress. The end connections of all members must be detailed so that they will be capable of taking the full stress of the members, and the truss must be braced against lateral displacement both as a whole and locally so that compression members that are considered of certain free lengths in the general design will be supported at these limits of length. In general truss members should be symmetrical about the plane of the truss, and the lines through the centers of gravity of the several members meeting at a common point should intersect in a common point.

Persuant of the author's intention to cover in this book only simple designing, this chapter will take up only the design of simple trusses and simple methods of finding the stresses in the same.

Plates I to III, inclusive, give co-efficients on the several truss members by which the stresses in these members may be found. The condition is that of a simple truss resting on walls and not of a truss acting to brace a building through the medium of knee braces. The trusses are further symmetrically loaded and not subject to unusual loads, such as suspended galleries, etc. To find the stress in any member compute the total load that a truss must carry; then multiply this by the co-efficient on the member in which the stress is desired.

The minus sign stands for compression, and the plus sign stands for tension.

As indicated on Plate III, these same diagrams may be used to find the stresses in a lean-to truss, that is, a truss of the shape of half of one of these. The stresses, for the same panel loads, will be the same for the half truss as for the full truss for all members except the horizontal member or the bottom chord and the long inclined member or the top chord. The "total load" for a lean-to truss is of course the load that the full truss would carry and not the load on the half truss. The co-efficient for each member of the bottom chord is reduced by .375, .433, etc., for the several pitches. It is seen that these are the stresses of the middle portion of this chord, which, of course, has a nominal stress when the truss is supported at the peak. (By using the term nominal it is meant to convey that there is no calculable stress in the member in question.) The top chord stress in the half trusses will be reduced throughout by the amounts given on Plate III.

Plates IV to VIII, inclusive, give the stresses, in terms of the panel loads P and the lengths of members, for trusses with parallel chords. The panel load for a four-panel truss is one-quarter of the total load carried by the truss; that for a five-panel truss, one-fifth; etc. At each end there is of course a half panel load. It is seen that these stresses are worked out on the assumption that the full load is applied at the top chord. If the load or any part of it is applied at the bottom chord, the compression in all vertical members will be diminished by just the amount of the panel load that is transferred to the bottom chord, (or the tension in the verticals will be increased by that amount) ; the stresses in diagonal members and chords will not be affected.

The stresses in Plates IV to VIII, inclusive, are for uniform load on the trusses; that is, they are for the ordinary case of roof trusses carrying their full load and not subject to unsymmetrical loading. These diagrams would not apply to floor trusses, where the full load may not be applied uniformly; for, while they would give the maximum chord stresses, the web stresses, particularly

near the middle of truss, would be quite different under partial loading with the same panel loads.

Plate IX shows the method of finding graphically the stresses in a common form of roof truss, whose upper chord is sloped. In this example the stress computation i. simplified by omitting in the diagram loads 3, 5, and 7 and concentrating the roof loads at 2, 4, 6, and 8. The graphic diagram is made as though the vertical members of the truss were omitted. Members 2-11, 4-13, etc., would have nominal stress. Members 3-12, 5-14, etc., would have a compression equal to the panel load at 3.

Very frequently graphical computation of stresses may be greatly simplified and expedited by assuming some unimportant members to be absent. The stresses in the main members are not greatly affected by this short-cut.

The method of proceedure in finding by the graphical method the stresses in a truss is as follows:

First find the panel loads, and mark the same on the diagram. Then find the reactions, and mark these on the diagram. In the case shown on Plate IX the panel loads are the vertical forces shown at 2, 4, 6, and 8.

(Note that loads 2 and 8 are 1½ single panel loads and 4 and 6 are equal to two single panel loads.)

The reactions are the forces shown at 10 and 18. Ordinarily the reactions are each equal to one-half the sum of the panel loads.

The next step is to letter the diagram of the truss. This is done by placing a letter below the truss, then at the ends of truss and between the panel loads, then in each triangle making up the frame of the truss. The object in this lettering is to make it possible to designate any member or force by naming two letters, one on each side of that member or force. Thus, in passing from the space A to the space B the reaction at the left end of truss will be crossed; that reaction is then the force AB; in passing from space B to space C the panel load at 2 is crossed; that panel load is then BC; in passing from space M to space L the member 12-4 is crossed; that

member then ML, etc. The significance of this lettering of the spaces can best be understood by a study of the subsequent processes.

The next step is to make a diagram of the applied loads. These loads are the two reactions and the several panel loads. These are drawn to scale. In the example on Plate IX, BA is the left-hand reaction and AF is the right-hand reaction. FE, ED, DC, and CB are the several panel loads. The arrows indicate the direction of these forces. If the reaction AB is 25,000 lbs., the line AB in the stress diagram will be made 25 units in length on some suitable scale. All stresses and forces will be laid out or measured on this same scale.

The stress diagram is then completed in the following manner. Beginning at either end of the truss, as at the right end, a line is drawn from A parallel to the member AG (or 16-18); then a line is drawn from F parallel to FG (or 8-18). The intersection of these lines is marked G. The length of the line FG, measured on the chosen scale, is the stress in the member FG, and the length of AG is the stress in member AG; the former is compression, and the latter is tension, as indicated by the minus and plus signs on the members. Lines EH and GH are drawn parallell to their respective members, locating the point H. Then HJ and AJ are drawn; then JK and DK; then KL and AL; then LM and CM; then MN and BN. If the final line BN is found to be parallel with member 2-10 the polygon is said to "close." This is evidence that the work is correct. The diagram could have been worked up from both right and left ends of the truss at the same time, as by drawing AN and BN, NM and CM, etc. The closing line would then be one of the short lines about L, K, and J. All of the stresses in the members are found by scaling this diagram.

In order to find the sign of a stress proceed as follows: Select a point, as 4, and trace the diagram of the forces meeting at this point. This diagram is CDKLM. If the direction of one of these forces is known, the direction

or sign of the others may be found thus. In this case the direction of CD is known, that is, this force is down or toward the point4. Following the diagram around in this direction the next force is CM, which is in the direction of the arrow, or toward point 4. The next force is ML, which is also toward point 4. The next force around the polygon is LK, which is downward to the right or away from point 4. KD is toward point 4. All the members that can be replaced by forces toward point 4 are in compression. LK or the force away from point 4, is in tension. In this manner the sign of any of the stresses can be found. It is necessary to try only a few points. The top chords will be in compression and the bottom chords will be in tension.

As in the case of the previously mentioned trusses the diagram of Plate IX is for uniform loading. However the method may be applied to find the stresses for any sort of loading. If the loading is unsymmetrical, the reactions will not be equal. These reactions may be calculated by taking moments around either support. The closing of the polygon of forces will check the correctness of the reactions as well as other parts of the work.

Plate X shows another style of roof truss and the graphical solution of the stresses in the same. The methods are the same as for Plate IX. Note that the top chord of this truss is member 3-5. Members 3-4 and 4-5 act merely as supports for the panel load at 4. The diagonals in the middle quadrilateral have nominal stress.

Plate XI shows a truss similar in shape to that on Plate X and a simplified method of finding the stresses in the main members. The applied loads are here concentrated at 2 and 3. If there are other members than these main members, they may be light enough to need no special calculation.

Plates XII and XIII show a detailed and a simplified method of finding the stresses in the truss shown, as also Plates XIV and XV.

Plate XVI shows still another common form of roof truss and the graphical solution of the stresses.

PLATE I
STRESSES IN ROOF TRUSSES

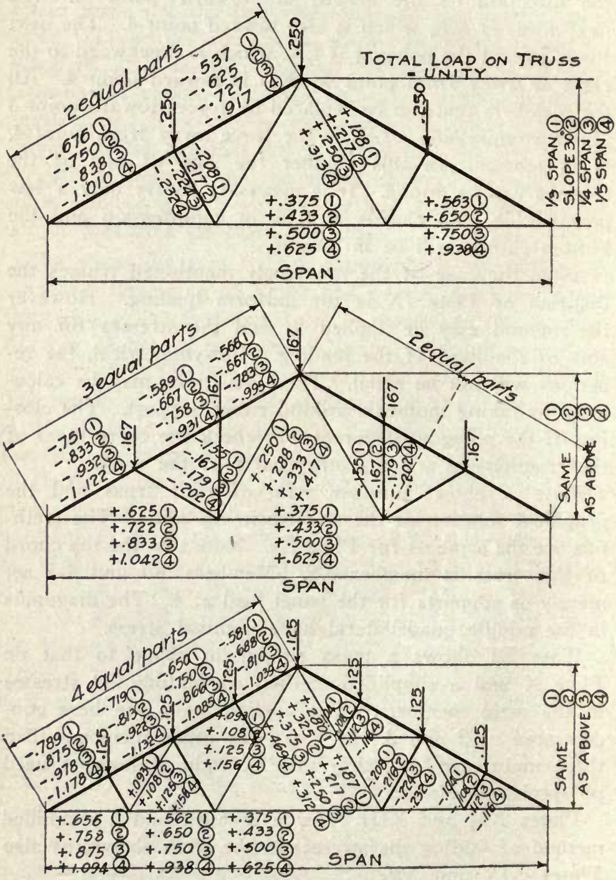


PLATE II
STRESSES IN ROOF TRUSSES

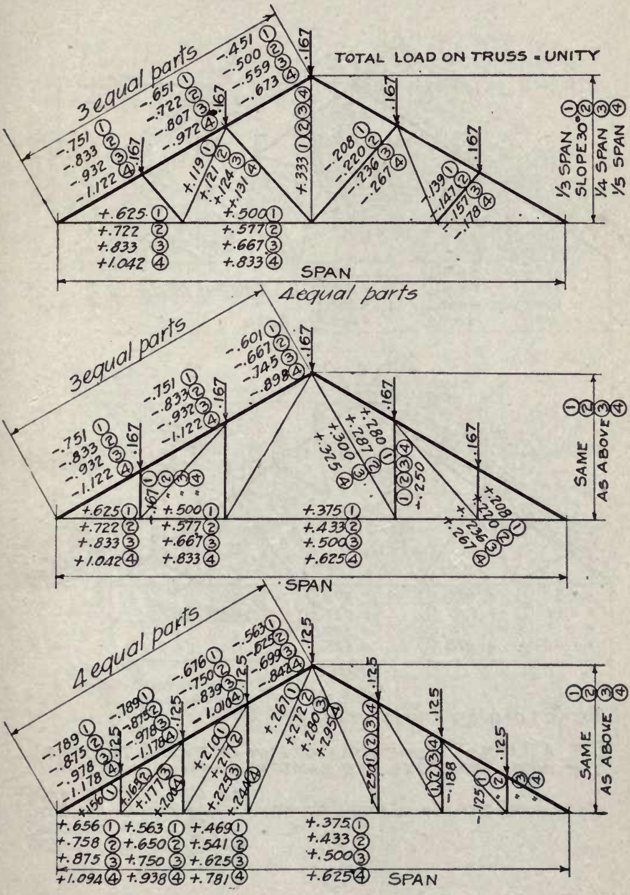
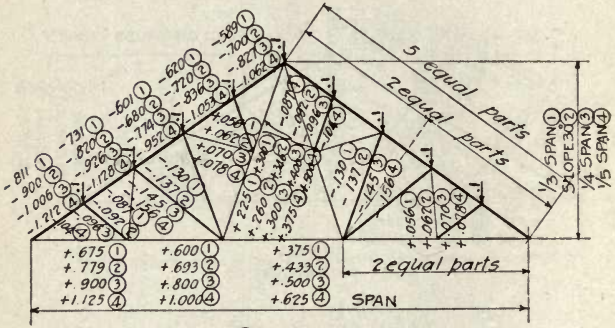
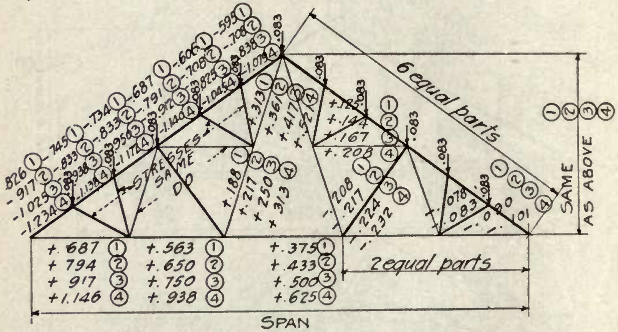


PLATE III
STRESSES IN ROOF TRUSSES





TOTAL LOAD ON TRUSS IS UNITY

FOR A LEAN-TO (ONE-HALF OF ANY OF THESE TRUSSES) THE WEB STRESSES ARE SAME AS GIVEN IN FIGS.

TOP CHORD STRESS IS REDUCED BY.	.451 ① .500 ② .559 ③ .673 ④	BOTT. CHORD STRESS BY	.375 ① .433 ② .500 ③ .625 ④

PLATE IV
TRUSSES WITH PARALLEL CHORDS

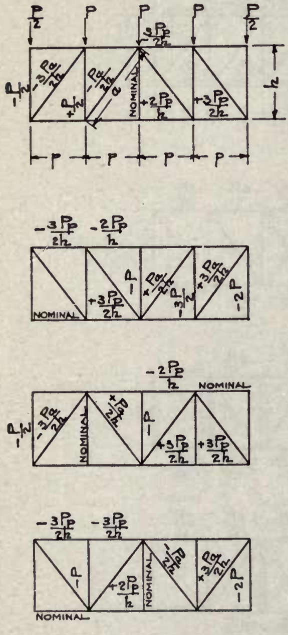


PLATE V
TRUSSES WITH PARALLEL CHORDS

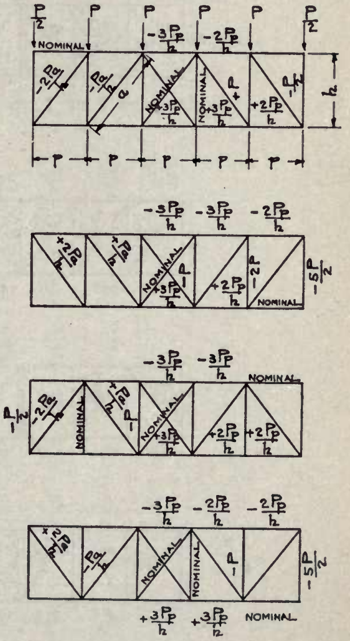


PLATE VI
TRUSSES WITH PARALLEL CHORDS

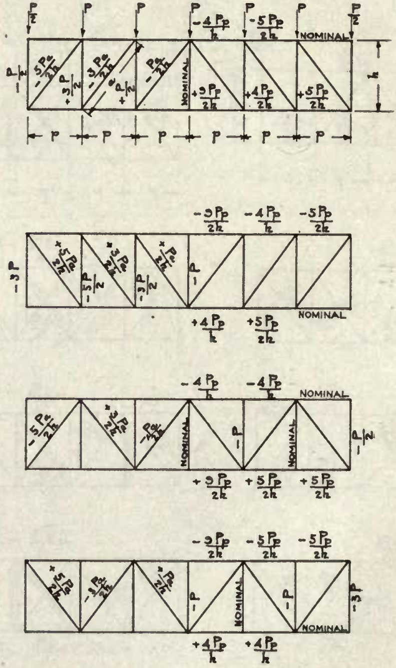


PLATE VII
TRUSSES WITH PARALLEL CHORDS

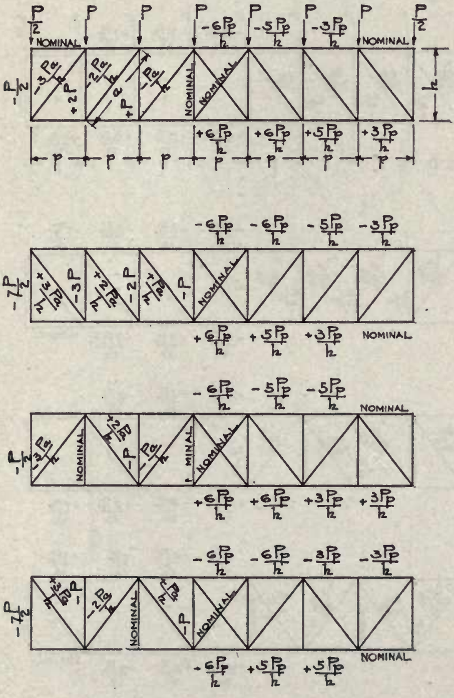


PLATE VIII
TRUSSES WITH PARALLEL CHORDS

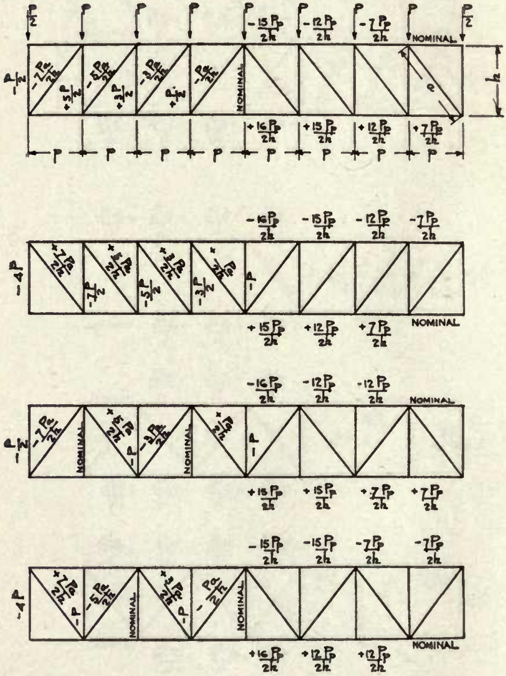
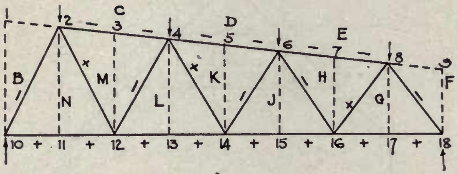


PLATE IX



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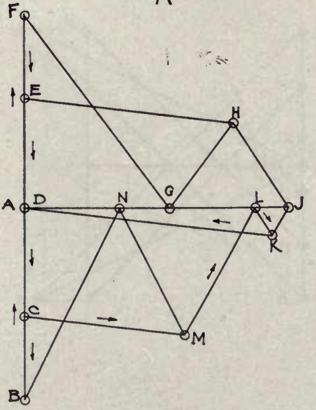

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PLATE X

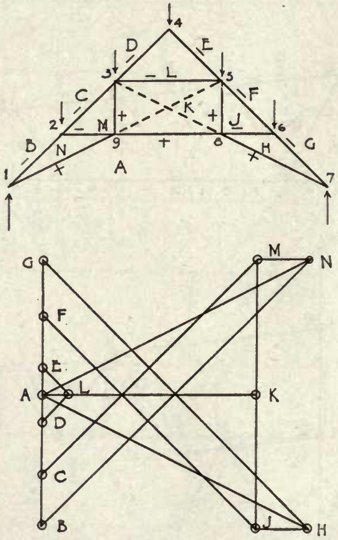


PLATE XI

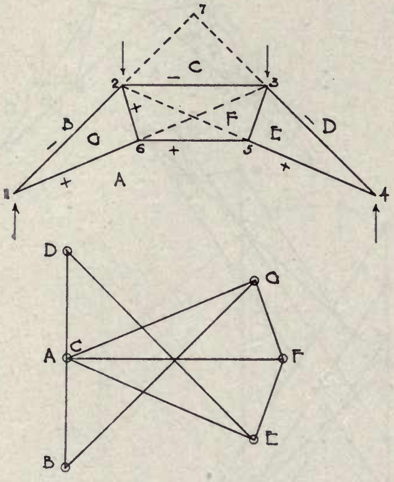


PLATE XII

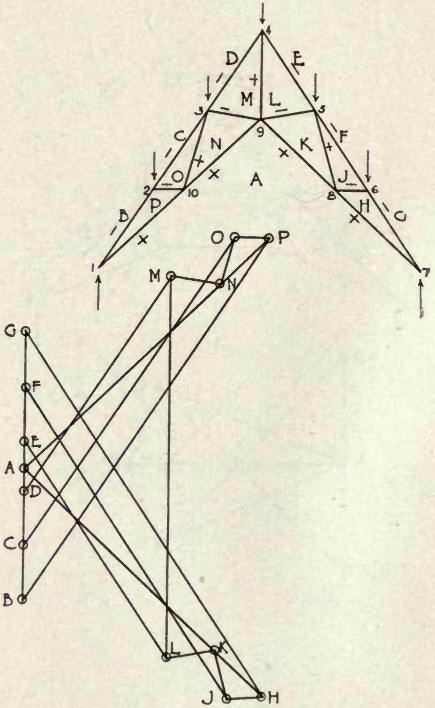


PLATE XIII

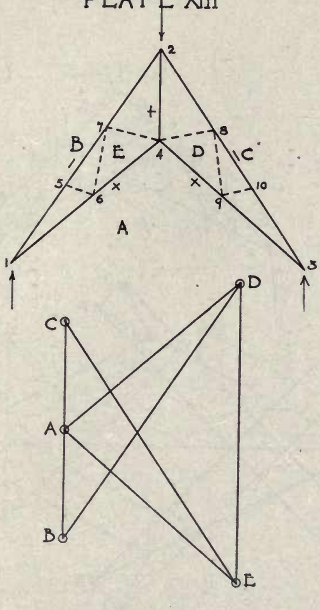


PLATE XIV

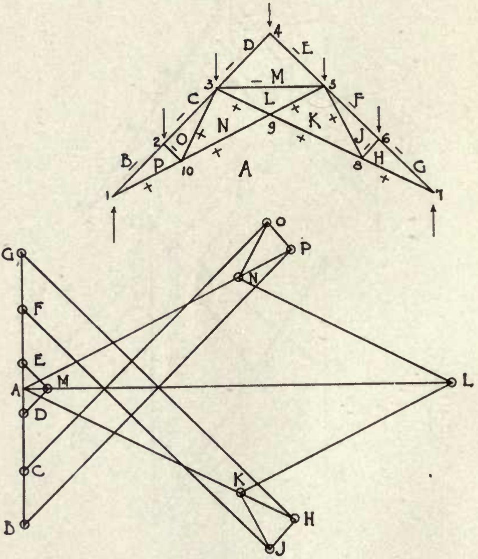


PLATE XV

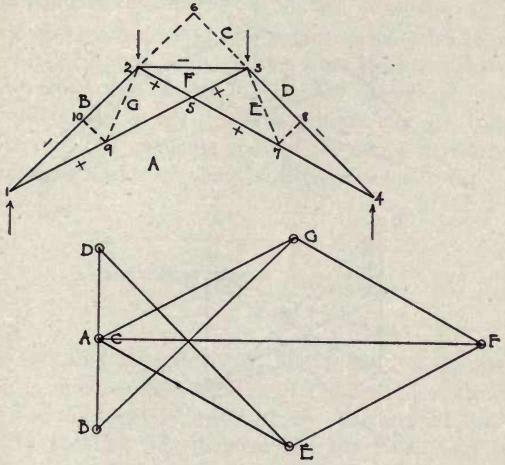
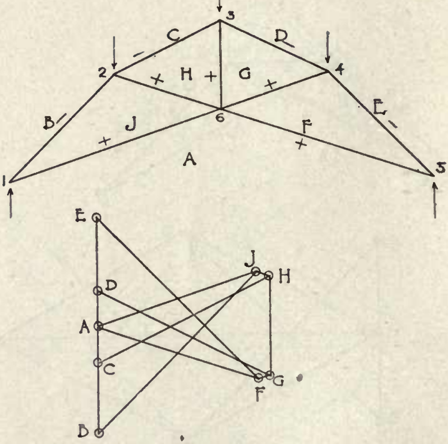


PLATE XVI



In riveted trusses light tension members are usually made of angles, single or double. Flats are sometimes used, but they are troublesome in a truss, because they are apt to be buckled when the truss is riveted up.

When a single angle is used in tension, its effective area should be counted as the area of one leg only of the angle. Thus, a 3"x3"x⅜" angle would be counted as though it were a 3"x⅜" flat, a 5"x3½"x½" angle would be counted as a 5"x½" flat, etc. This is to compensate for the lack of symmetry, or the eccentric application of the stress, and the consequent bending stress in the member.

A member composed of two angles symmetrically placed with respect to the plane of the truss does not have the bending stress mentioned in the last paragraph. However such angle is not good in tension for its full sectional area. The available area of the angle is reduced by the punching away of metal for rivet holes.

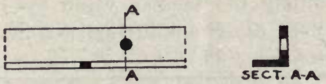


Fig. 1.

It is seen that at section AA, Fig. 1, the area of metal in tension is equal to the full area of the angle less the product of the thickness of metal by the diameter of the rivet hole. In practice this diameter of the rivet hole is assumed to be ⅛" greater than the nominal diameter of the rivet. It is also seen that if the hole in the other flange of the angle is near the section AA, the net area through a zig-zag line cutting both holes may be less than through the square section cutting one hole only. In detailing tension members care must be taken to see that the minmum number of rivet holes occur at or near any given transverse section. Angles having three or four rows of rivets (as 6"x3½" and 6"x6" angles) usually have two rivet holes deducted in the net section.

In channel sections in tension one or more rivet holes will be deducted, depending on the deail of the member. If the member has lattice or batten plates, that is, if the flanges are punched, two flange holes will be deducted from the gross area and as many web holes as occur in the same transverse section.

Eye-bars and rods with loops at the ends are designed for tension in the full section of the bar or rod, as the eye or loop is made capable of taking the full value of the bar or rod.

Bolts or rods with either plain nuts or clevis nuts at the ends are designed for tension in the full section of the bolt or rod, provided the threaded ends are upset. If the threaded ends are not upset, the value of the bolt or rod is only that of the metal in a circle whose diameter is measured at the root of the threads. Table I gives the tensile strength of rods of various diameters, the area being measured at the root of threads. The unit used in the table is 10,000 lbs. For any other unit, as 16,000 lbs. per sq. in. multiply the tabular value by 1.6, etc. The screw threads used are Franklin Institute Standard. (See Godfrey's Tables, page 35.)

TABLE I.

Tensile Strength of Rods at 10,000 lbs. per sq. in. Area Measured at Root of Threads.

Dia. of Rod in In.	Tensile Strength.	Dia. of Rod in In.	Tensile Strength.	Dia. of Rod in In.	Tensile Strength.
3/4	3020	2 1/8	26500	3 1/2	75500
7/8	4200	2 1/4	30240	3 5/8	81700
1	5500	2 3/8	34210	3 3/4	86400
1 1/8	6940	2 1/2	37160	3 7/8	93000
1 1/4	8910	2 5/8	41550	4	99900
1 3/8	10570	2 3/4	46180	4 1/8	107100
1 1/2	12950	2 7/8	51070	4 1/4	113300
1 5/8	15150	3	54290	4 3/8	120900
1 3/4	17440	3 1/8	59570	4 1/2	127400
1 7/8	20480	3 1/4	65100	4 5/8	135500
2	23020	3 3/8	70900	4 3/4	142200

In building work a unit stress of 16,000 lbs. per sq. in. is usually allowed on rods and bars. The same unit is sometimes allowed on the net section of shapes such as angles and channels, though 15,000 lbs. is preferable, because of the uncertain effect of punching, and because stress is not so uniformly distributed in shapes as in rods and bars.

Examples:

(1) Required the section of a tension member in a light truss to take 11,000 lbs. of stress. Here the area, at 15,000 lbs. is .73 sq. in. The area of one leg of a 3"x2½"x ¼" angle is .75 sq. in. This could be used. A 1⅛" rod, not upset has a tensile strength, by Table I, at 16,000 lbs. per sq. in., of 11,100 lbs. This rod could be used, if the style of truss permit.

(2) Required the section of a member to take a tensile stress of 35,000 lbs. The net area required, at 15,000 lbs. per sq. in., is 2.33 sq. in. If the member is composed of 2 angles, each angle will have a net area of 1.17 sq. in. or a gross area, adding .33 sq. in. for the rivet hole, of about 1.50 sq. in. A 3"x3"x¼" angle has a net area of 1.44—.22=1.22 sq. in. (The deduction of .22 is for a ¾" rivet hole or ⅞x¼.) Two such angles could be used.

(3) Required the section of an upset rod to take a stress of 80,000 lbs. The area, at 16,000 lbs. per sq. in., is 5 sq. in. By reference to a table of the area of rounds (Godfrey's Tables, page 61, et seq.) it is found that a 2 9-16" round rod would be required. If two rods were used, each should have a diameter of 1 13-16."

Tension members in timber trusses are usually made of steel or iron rods, though the bottom chords are often made of wood. The section required is usually determined by the detail at the ends or splices. Wooden members do not admit of very efficient details for tension. A large portion of a wooden tension member may be notched away for the splice or bored out for bolts. A tensile stress of 1,200 lbs. per sq. in. for white pine and 1,600 lbs.

per sq. in. for yellow pine or white oak may be allowed on the net section of the wood.

COMPRESSION MEMBERS.

The selection of the size of compression members in a truss should be carried out by the methods of Chapter IV. Tables IV and V of that chapter, as stated in the chapter, are for single angles as members having square-ended or rigid details. In an ordinary light truss, if a single angle is used in compression, only about half of the value shown in Tables IV and V should be used as safe values of the members.

Tables VI to IX, inclusive, of Chapter IV may be used for truss members without any reduction of the tabular load.

In general it is best to select standard angles and a small number of different sizes for any given truss.

TRUSS MEMBERS IN BENDING.

Truss members are sometimes subject to transverse or bending stresses as well as direct stress (tension or compression). Such members must be designed for both bending and direct stress, the unit stress in the steel being kept within certain limits.

When all of the load on a truss is cencentrated in beams that connect to the truss at the panel points only, there will be no bending in the truss members, but dirsct stress only. The roof load, however, is very frequently distributed uniformly along the top chord of a truss or concentrated in beams or purlins that do not connect to the truss at panel points. The top chord must then act as a beam as well as a compression member and must be designed accordingly. A member suitable for this condition is deep vertically. Examples of such members are: two angles of unequal legs with the long legs vertical, two channels, two angles of equal legs with a deep plate riveted between them. In wooden trusses of course the member is made deeper in the vertical dimension than in the horizontal.

There is much variation in the practice of designing members under combined direct and bending stress. There is also very frequently little attention paid to the necessity for care in such designing. The rigid or correct treatment of the problem will not be given here, as it involves structural engineering principles outside of the scope of this book. Approximate methods only will be given here. They will be found to be safe, though not wasteful; the results will be close to correct theoretical methods and very much superior to the guess-work so often resorted to.

WOODEN TRUSS MEMBERS IN BENDING.

First find the actual or equivalent uniform load on the member by the methods of Chapter VI. Then find the value of C, that is, the product of the uniform load by the span in feet. The span in feet is the horizontal distance between the panel points or supports of the member considered. (It is not the inclined distance for inclined members.) Next find in Table I of Chapter VI a section whose value C is greater than that just computed, for a trial design. Then find by Chapter IV, using Table I, the value of this member in compression. Now compare the value of C required with that of the member selected as also the actual compression in the member with its allowed compression, and add these two ratios; they should equal unity. Thus, if the member is under 7-10 of its allowed bending, it may carry at the same time 3-10 of its allowed compression. If the sum of the ratios is greater than unity, select a heavier or deeper member; if less than unity, select a lighter section.

Examples:

(1) Required the section of a rafter five feet long on the slope and four feet in the horizontal direction, carrying a load of 400 lbs. per horizontal foot and subject to 10,000 lbs. of compression. In this case C is $400 \times 4 \times 4 = 6{,}400$. A4"x6" in white pine has a value $C = 12{,}800$, by Table I, Chapter VI. The rafter would be stayed horizontally by the joists resting upon it, hence the unsup-

ported dimension would be six inches. The ratio of this to the length of rafter is 10. By Table I, Chapter IV, the allowed unit stress for this ratio is 820 lbs. per sq. in. The member is then good for a compressive stress of $820 \times 24 = 19,680$ lbs. The member is thus subject to .50 of its allowed bending value and .51 of its allowed compression. The sum of these two is 1.01, and the member is therefore correct.

(2) Required the size of a horizontal chord member 10 ft. between panel points, the roof load being 600 lbs. per foot and the compression being 28,000 lbs. The roof beams are five feet apart, that is, at panel points and midway between panel points. In this case $C = 600 \times 10 \times 10 = 60,000$. In yellow pine a 4x16 piece has a value $C = 113,770$. For compression the unsupported length is 5 ft. and the width is 4 in. The ratio for Table I, Chapter IV, is 15 and by interpolation the unit stress is found to be 730 lbs. per sq. in. The allowed compression is $730 \times 4 \times 16 = 46,720$ lbs. The bending is then .53 of the capacity and the compression .60. This gives a total of 1.13 which is more than the limit. The member could be 5″x16″, or by trial it will be seen that 6″x14″ would be somewhat stronger than necessary. This could be made of three 2″x14″ pieces spiked or bolted together.

STEEL TRUSS MEMBERS IN BENDING.

The same method of proceedure would be used for steel members as for wooden members except that the load is found in tons and Q instead of C thus found.

Examples:

(1) Required the section of a top chord member 4 ft. long, the compression being 75,000 lbs. and the load per ft. on the chord 1,000 lbs. Here the load per panel on the chord is 4,000 lbs. or 2 tons and Q is 8. In Table V, Chapter VI, it is seen that Q for two angles 6″x4″x⅜″, with the long legs vertical, is $2 \times 17.7 = 35.4$. In Table VI, Chapter IV, it is seen that this same section 4 ft. long has a strength in compression of 98,000 lbs. 75,000 divided by $98,000 = .77$, and 8 divided by $35.4 = .23$. The sum

of these two ratios is just unity; hence this section is correct.

(2) Given a top chord supporting a reinforced concrete slab. Panel length, 12 ft.; compression, 80,000 lbs.; load per ft., 1,600 lbs. The total load on a panel is $12 \times 1{,}600$ lbs. or 9.6 tons, and Q is 115.2. By Table II, Chapter VI, Q for 2-12″ channels 20.5 lbs., is 228.1. By Table XVI, Chapter IV, the same channels in compression for a length of 12 ft. will carry 161,000 lbs. The sum of the two ratios will be found to be close to unity. It is to be noted that this channel section would not be good for 161,000 lbs. if it were not supported continuously or at close intervals laterally, or unless the channels were separated and latticed, as indicated in Table XVI, Chapter IV.

A common method of providing for bending in the top chord of a roof truss is by using a web plate between two angles, such sections as shown in Godfrey's Tables, page 122. By using the section modulus as found in that table the stress in such member due to bending may be found. An approximate method is as follows:

Find the size of a pair of angles that will take the compression, acting alone; then fiind the size of a web plate which at 16,000 lbs. per sq. in. will take the bending.

The following table will facilitate the selection of a web plate to take the bending stress.

TABLE II.

Capacity of Steel Plates in Bending Fiber Stress 16,000 lbs. per sq in.

Q is product of span in ft. and unif. load in tons.

Size of Plate.	Q	Size of Plate.	Q	Size of Plate.	Q
6x¼	8.0	11x⅜	40.3	16x⅜	85.3
6x5/16	10.0	11x½	53.8	16x½	113.8
7x¼	10.9	12x⅜	48.0	17x⅜	96.3
7x5/16	13.6	12x½	64.0	17x½	128.5
8x¼	14.2	13x⅜	56.3	18x⅜	108.0
8x5/16	17.8	13x½	75.1	18x½	144.0
9x¼	18.0	14x⅜	65.3	19x7/16	140.4
9x⅜	27.0	14x½	87.1	19x9/16	180.5
10x⅜	33.3	15x⅜	75.0	20x7/16	155.5
10x½	44.4	15x½	100.0	20x9/16	200.0

Examples:

(1) Required the section of the top chord of a roof truss; panel length, 6 ft.; load per ft., 800 lbs.; compression, 28,000 lbs. By Table VI, Chapter IV, it is seen that 2 angles 3"x2½"x¼" will take the stress of 28,000 lbs. in a length of 6 ft. The load on the chord section is $800 \times 6 = 4{,}800$ lbs. or 2.4 tons. Q is $2.4 \times 6 = 14.4$. By Table II, this chapter, it is seen that an 8"x¼" plate will take the bending.

(2) Required the section of the top chord of a roof truss, the length of panel being 12 ft. along the slope and 10 ft. horizontally. Load per ft. along the slope, 2,000 lbs.; compression 68,000 lbs. By Table VI, Chapter IV, it is seen that 2 angles 6"x3½"x⅜" will take the compression. The load on the chord section is $12 \times 2{,}000 = 24{,}000$ lbs. or 12 tons. Q is $12 \times 10 = 120$. (Since 10 is the span and not 12.) An 18"x7-16" plate has a value Q= 126.

CHAPTER IX.

Floor Arches and Slabs.

Table I, from "Cambria Steel" gives the weight and safe load per square foot for hollow tile floor arches.

TABLE I.

SAFE LOAD IN LBS. PER SQ. FT. ON HOLLOW TILE FLOOR ARCHES (INCLUDING WT. OF TILE AND FLOOR.)

Depth in In.	Wt. of Arch per Sq. Ft. Lbs.	Span of Arch in Feet. 3	4	5	6	7	8
6	27	336	189	121	..	...	..
7	29	429	242	155	..	..	..
8	32	523	294	188	131	..	..
9	36	616	347	222	154	113	..
10	39	709	399	255	177	130	100
12	44	896	504	323	224	165	126

In order to realize this safe load the beams must have tie rods, so that the thrust of the arch will not come against the unstiffened beam. Tie rods in hollow tile construction are very often inadequate, as a little calculation will show. Take for example a 10-inch arch on a 6-ft. span. Assuming a total load of 150 lbs. per sq. ft. and an effective depth of the arch of 6 inches, the thrust per foot of the arch is found to be 1350 lbs. It is common to see ¾-in. tie rods spaced 6 or 8 ft. apart in beams of this size. At 6 ft. the stress on a rod is 8,100 lbs. or about 33,000 lbs. per sq. in. on the rod at root of threads. Besides this heavy stress on the rod the side force on the flange of the beam is too great, and rods should be closer to stiffen the flange. In addition to this the rods are very often placed at the middle of the web of the beam, whereas they should be near the bottom flange, say about 3 inches from the bottom of the beam.

It is true that the thrust of one arch will balance that of the next one, but this does not apply to the last arch of a row, where a single tie rod is often made to take the full thrust of the floor arch.

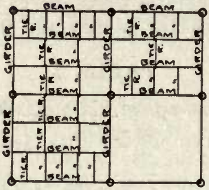


Fig. I.

A good practice, and one that ought to be generally adopted, is to double the number of tie rods for all outer arches (as shown in Fig. 1) whether these are adjacent to a stair well or other opening in the floor or against a wall. The side of a brick wall is not a suitable abutment for a floor arch; furthermore, the outside channel or beam is not always in contact with the wall. By this method the rods in interior arches may be 6 or 8 ft. apart, while those in outer arches will be 3 or 4 ft. apart.

Tie rods are usually ⅝", ¾", and ⅞" rods. They are ordered about three inches longer than the distance center to center of beams.

Tile arches are not suitable for wide spans between the beams. The upper limit should be about 7 or 8 feet. In wide spans the compression in the tiles becomes great, and the manner in which these tiles are laid does not inspire confidence as to their ability to resist heavy compressive stresses. What are called end construction tiles, the most common in use, have their thin webs butting together and fitting very imperfectly. The filling of the joints with mortar is still more imperfect.

Reinforced concrete floor slabs are commonly made in thicknesses of about 3 to 7 or 8 inches. The principles of reinforced concrete design laid down in Chapter VI apply also to slab construction. On account of the large predominance of concrete over steel and the resultant stiffness of the slab and on account of the fact that tension in this concrete is ignored, it is safe to use 16,000 lbs. per sq. in. as the calculated working stress in the steel. The safe compressive stress of 600 lbs. per sq. in. will be used in the concrete. This would give a steel area of .94 per cent. of the area of the slab, when the balance between steel and concrete is effected.

The span of a slab is to be taken as the clear distance between beams or other supports, and the slab is taken as a simple beam for this span. No allowance for supposed continuity of slabs is made.

The bending moment on a slab, by the above standard is as follows:

$$M = 106\ D^2$$

where M is the bending moment in ft.-lbs. per ft. width of slab and D is the depth of the slab in inches.

Table II is worked out on the basis of the above formula. The steel reinforcement is given in square rods. Round rods or a steel mesh having the same sectional area per foot width of slab could be used. The steel reinforcement should lie about one-eighth of the depth of slab from the bottom.

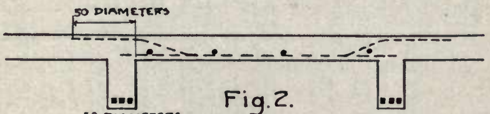


Fig. 2.

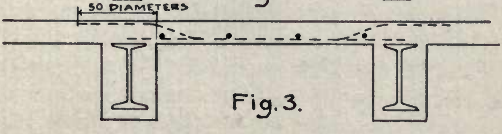


Fig. 3.

Generally every alternate bar or every third bar should be bent up and run beyond the support, as indicated in Figs. 2 and 3. This is to prevent cracking in the upper part of the slab at the supports. Of course when two slabs come together on a beam, these extended rods will overlap. This is not shown in Figs. 2 and 3 because of the confusion that it would entail in these sketches.

Besides the main reinforcement in a slab there should be transverse reinforcement. This may be made up of ¼-in. to ½-in. square or round rods. In heavy slabs ½-in. rods may be spaced 2 ft. apart. In light slabs ¼-in. rods may be spaced one ft. apart. These rods are to prevent shrinkage cracks in the slabs.

For maximum economy in weight, as in a high building, the slabs and spans should be about in the relation of those in Table II. There are many circumstances in which it is economical to use a deeper slab than those shown in the table, in which case less steel reinforcement can be employed. The reason for this is that a large part of the

TABLE II.

Maximum Span in Clear between Supports for Reinforced Concrete Slabs.

Depth of Slab in Inches.	Reinforcement: Dia. of Sq. Rods in In.	Reinforcement: Distance C. to C. of Rods in In.	Maximum Span in Feet for Total Uniform Load per Sq. Ft. of 100 Lbs.	125 Lbs.	150 Lbs.	175 Lbs.	200 Lbs.	250 Lbs.	300 Lbs.
2½	¼	2.7	7.3	6.5	5.9	5.5	5.1	...	...
3	¼	2.2	8.7	7.8	7.1	6.6	6.2	5.5	...
3½	⅜	4.3	10.2	9.1	8.3	7.7	7.2	6.4	5.9
4	⅜	3.7	11.6	10.4	9.5	8.8	8.2	7.4	6.7
4½	½	5.9	13.1	11.7	10.7	9.9	9.3	8.3	7.6
5	½	5.3	14.6	13.0	11.9	11.0	10.3	9.2	8.4
5½	⅝	7.6	16.0	14.3	13.1	12.1	11.3	10.1	9.2
6	⅝	6.9	....	15.6	14.3	13.2	12.4	11.1	10.1
6½	⅝	6.4	....	16.9	15.5	14.3	13.4	12.0	10.9
7	¾	8.5	....	18.2	16.6	15.4	14.4	12.9	11.8
7½	¾	8.0	....	....	17.8	16.5	15.4	13.8	12.6
8	¾	7.5	....	....	19.0	17.6	16.5	14.7	13.5

cost of a reinforced concrete slab is in the forms. An inch or so more of concrete does not make much difference in the cost, and it may effect considerable saving in the steel.

It is plain that if a deeper slab than that shown in the table is used, the stress in the concrete will be less as also that in the steel. The concrete of course cannot be varied, but the steel reinforcement may be reduced. The stress in the steel reinforcement will be directly proportional to the total load per sq. ft. for a given span and depth of slab. The table may then be used to find the amount of steel needed for a given span and depth and a load different from that in the table, as follows:

Given a span 8.2 ft. in the clear and a slab 4 in. deep, to support a total load of 150 lbs. per sq. ft. By Table II it is seen that this slab would carry 200 lbs. per sq. ft. with the reinforcement shown in the table. For a load of 150 lbs. per sq. ft. the reinforcement would need but ¾ of the standard area, or the rods may be spaced 4/3 as far apart. Four-thirds of 3.7 inches is 4.93 in., or say 5 inches. Square rods ⅜ in. in diameter and spaced 5 in. would then be used.

Designers and constructors, particularly the latter, in dealing with reinforced concrete slabs make many grave errors in the matter of framing around openings in the floor. It is common but very bad practice, where openings as to be left in a floor slab, to cut off the reinforcing rods at the edge of the opening and to use so-called headers, that is, rods parallel to the side of the opening. This is an idea borrowed from the practice in wooden joist framing, but the user of this idea ignores the fact that in wooden joist framing the header is carried by double joists.

Fig. 4 shows the common but erroneous methods of taking care of openings in a floor slab. The rods are all laid nicely in parallel lines, and they look well to anyone ignorant of their office. The arrangement shown in Fig. 5 is not nearly so neat, but the main rods reach to sub-

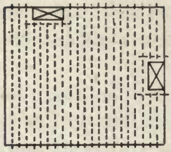


Fig. 4.

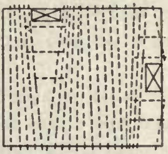


Fig. 5

stantial bearings in every case; they do not throw their load on some other rod already burdened with its full share.

The sides of the rectangles in Figs. 4 and 5 represent walls or beams supporting a slab. The cross rods in Fig. 5 are not "headers," but are merely for the purpose of reinforcing locally the slab in the triangular space. Expanded metal or other steel mesh could be used in these triangular spaces.

Another error in laying floor rods around openings is to place the rods parallel up to the opening and then to bend or curve them in plan around the opening. This is as bad as the arrangement of Fig. 4. Reinforcing rods should not be bent or curved horizontally. Rods should run straight from support to support. If the opening is large, special beam framing should be made around it.

In tile-filled ribbed floors, if a rib must be omitted on account of an opening, the adjacent ribs should make up in extra thickness and reinforcement.

CHAPTER X.

Structural Details.

No attempt will be made in this book to cover all kinds of structural details, for the reason that the book is not one that aims to cover structural designing in all its branches, only simple riveted structural work being considered. Details of pin connected members will be omitted entirely. As stated in the introduction, the book is intended to cover only simply design as applied to structural parts of a building.

Rivets. The strength of a rivet has two phases, as exhibited in the two ways in which it may fail. First the rivet may fail in shear, or by cutting the shank in the plane of the surfaces of the metal joined. Next it may fail in bearing, or by crushing against the metal. When a rivet fails in shear, it is cut in two by excessive strain, such as would result from the action of shear knives. When it fails in bearing, the metal of the rivet crushes against the side of the hole, allowing the parts that are joined by the rivet to slip.

The strength of a rivet in shear is measured by the area of steel that it is necessary to cut in the shearing off of the rivet, that is, by the area of the cross section of the rivet shank. This is always taken as the area of the cross section of the rivet before driving.

The strength of a rivet in bearing is measured by the projection of the semi-intrados of the rivet hole in the plate, that is, the product of the diameter of the rivet and the thickness of the plate. Here, too, the nominal diameter of the rivet, and not the diameter of the hole, is used.

The unit stresses that may be allowed in rivets vary with the kind of work, those for railroad bridges being low and those for quiescent loads, such as buildings, be-

ing higher. Units of 10,000 lbs. per sq. in. for shear and 20,000 lbs. per sq. in. for bearing are very often used. Better units are 9,000 and 18,000 respectively Table I gives the safe value of rivets on the basis of these two sets of units for the sizes of rivets generally used in building work.

TABLE I.

Shearing and Bearing Valve of Rivets.

All Dimensions in Inches.

Diam. of Rivet.	Sgle. Shr. at 9000 Lbs.	Bearing Value for Different Thickness of Plate at 18,000 Pounds per Square Inch.										
		¼	5/16	⅜	7/16	½	9/16	⅝	11/16	¾	13/16	⅞
⅝	2760	2810	3520	4220	4920	5630	6330	7030	.....	.....	.....	....
¾	3980	3380	4220	5060	5910	6750	7590	8440	9280	10130	.....	....
⅞	5410	3940	4920	5910	6890	7880	8860	9840	10830	11810	12800	1378
1	7070	4500	5630	6750	7880	9000	10130	11250	12380	13500	14620	1575
Diam. of Rivet.	**Sgle. Shr. at 10000 Lbs.**	**Bearing Value for Different Thicknesses of Plate at 20,000 Pounds per Square Inch.**										
		¼	5/16	⅜	7/16	½	9/16	⅝	11/16	¾	13/16	⅞
⅝	3070	3130	3910	4690	5470	6250	7030	7810	.....	.....	.....	....
¾	4420	3750	4690	5630	6560	7500	8440	9380	10310	11250	.....	....
⅞	6010	4380	5470	6570	7660	8750	9840	10940	12030	13130	14220	1531(
1	7850	5000	6250	7500	8750	10000	11250	12500	13750	15000	16250	1750

Bolts. If bolts are used in punched holes, take two-thirds of the values in the table. If turned bolts in tight-fitting reamed or drilled holes, the bolts having ¼-in. washers, so that no part of the thread is in the hole, the values of the table may be used.

The strength of the connection shown in Fig. 1 is the single shear value of two rivets, for evidently these two rivets must shear before the connection can fail. But the strength of the connection is also that of the two rivets in bearing either against the plate or the angle, for if this

metal is too thin, it will be crushed by the pressure of the rivet. By reference to Table I it can be readily seen which is less, bearing or shear, and hence which is the real gage of the strength of the joint. Suppose, for example that the metal of the angle is ¼ in. thick and the rivets are ¾ in. in diameter. The value of a rivet in single shear is 3,980, and in bearing 3,380. The latter value governs. If the rivets were ⅝ in., single shear would govern, at 2,760. Note that the values in the table between the heavy zig-zag lines are greater than single shear and less than double shear.

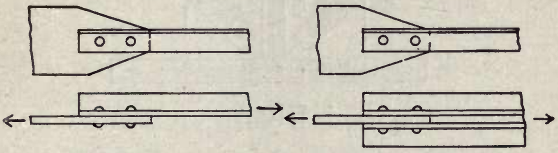


Fig. 1. Fig. 2.

The strength of the connection shown in Fig. 2 is the double shear value of two rivets, or four times the single shear value of one rivet, for to fail in shear each of these rivets must be sheared twice. The strength of the connection is also that of two rivets in bearing against the plate or the double thickness of angles. If, for example, a ⅝-in. plate be used and ⅜-in. angles, the thickness of plate will govern so far as bearing is concerned, and in ⅞-in. rivets the strength is 9,840×2 or 19,580 lbs. Double shear on two rivets is good for 5,410×4, or 21,640 lbs. The former value governs. If the plate were 11/16 in. or more in thickness, shear would govern. In ¾-in. rivets shear would govern with the ⅝-in. plate. This is indicated by the zig-zag line of Table I.

The foregoing rules and principles apply for finding the strength of the end connections of tension or compression members, or the strength of tension splices, or the strength of the end connection of beams and girders.

The rivets in any riveted connection should be symmetrically disposed about the line of application of the stress, insofar as it is practicable to effect this condition. This is to avoid eccentric stress on the rivets. If it is necessary to place rivets unsymmetrical with respect to the line of stress, additional rivets must be used.

No rivet connection should be made with less than two rivets, preferably not less than three.

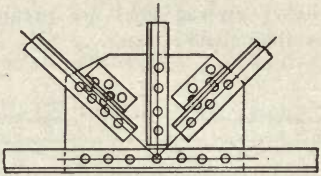


Fig. 3.

Frequently, in order to cut down the size of the gusset plate, lug angles are used to take some of the rivets in the end connection of a member as shown on the diagonal members of Fig. 3. Generally the larger number of rivets should be in the member itself.

Tension Splices. Splices in tension members should be made with splicing pieces having a net sectional area through any cross section (whether at right angles, diagonally, or zig-zag across the section) equal to the net sectional area of the piece cut. There must be rivets enough on each side of the cut to take the full stress in the member spliced.

Compression Splices. Splices in compression members are generally made by planing the ends of the members square, so that they will fit exactly one on the other and providing a sufficient number of splice plates to hold these planed ends rigidly in line.

In building columns made of I-shaped sections there should be a plate on the outside of each flange with about

six rivets above and below the cut in each plate. There should also be a plate on each side of the web of the column.

In columns made of two channels and two plates it is preferable to use a horizontal plate besides the splices on the cover plates. The reason for this is that the webs of the channels may not be opposite one another, and splicing plates on these webs cannot be riveted, as the section is a closed one.

Wherever there is a change in the general size of a column there should be horizontal plates used in the splice, so as to distribute the load of the upper column into the lower.

When only a portion of a compression member is cut and spliced, the full area and the full number of rivets should be used in the splice, even though the spliced part has the ends milled for a bearing; for in building up such piece in the shop the milled ends may not be in contact. It is practically impossible to insure close contact.

End Connections of Beams. The end connections of beams are commonly made according to the standards found in the Carnegie Pocket Companion (or Godfrey's Tables, pages 37 and 38).

Channels should have the same symmetrical end connection as beams of the same depth. Where this is not practicable, a 6″×6″ angle may be used with two rows of rivets in each leg.

Beams connecting to columns are usually supported on a shelf angle riveted to the column and are riveted through the flange to the same. An upper angle, shipped loose with the column, is riveted in the field to the top flange of the beam and to the column.

When more than four rivets are required to carry a beam or a girder on a shelf, stiffener angles are used to take the additional rivets. These should be placed with the outstanding legs directly under the beam.

End Connections of Girders. When a girder rests on a support such as the top of a column or a shelf having stiffeners under it, the metal of the column or of stiffener angles or diaphragms in the head of the column or the stiffener angles below the shelf should be directly opposite the metal of the end stiffeners of the girder. This is an important feature of design that is very often overlooked. It is illustrated in Fig. 4. If the end angles of this girder were turned with the outstanding legs at the end of girder, these angles would not be opposite the metal of the channel of the column. The result would be excessive bending either in the top plate of the colmun or in the flange angles of the girder.

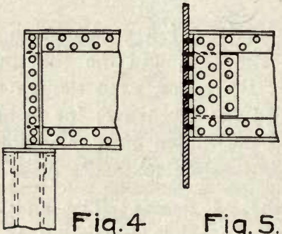


Fig. 4 Fig. 5.

When the end connection of a girder is with angles connecting to the web of the girder, there must be enough rivets through the web of the girder to take the full end reaction of the girder. These rivets are in bearing on the web of the girder, even though some of them pass through the flanges of the angles also. If there is not room enough for the required number of rivets, on the basis of this bearing value in the web, the fillers can be extended as in Fig. 5. The four additional rivets shown in this figure unite these fillers and the web plate so as to increase the value of the five rivets in the angles.

The field rivets in the girder connection of Fig. 5 will have a strength of 14 rivets in single shear or in bearing either on the angles or the metal to which they connect.

Seven-eighth-inch rivets are used in flanges as narrow as 3 inches; ¾-in. rivets, in flanges as narrow as 2½ inches; ⅝-in. rivets, in flanges as narrow as 2 inches. When it is known that ¾-in. rivets are to be used, the design must be made with this fact in view and flanges less than 2½ in. wide must not be placed where rivets will have to be driven in them. The same must be observed with other sizes. It is preferable, because of economy in the shop, to use only one size of rivet in a piece of work. An exception may be made in the case of channel flanges, as these must often take smaller rivets than the rest of the work. They must be handled twice in any event to punch web and flange holes, as these require separate dies.

Rivets should be spaced not less than three diameters apart center to center, nor generally more than six inches apart. They should not be closer to the edge of metal than about two diameters (two times the diameter of the rivet).

Lattice bars for single lacing should be about 60 degrees with the axis of the member. Lattice bars for double lacing should be about 45 degrees with the axis of the member. Some common sizes of lattice bars, with the depth of member in which they may be used are given in the following list:

Size of bar.	Depth of member.	Size of rivets.
1½x ¼	6 in. and under	⅝
1¾x ¼	7 to 8 in.	⅝
2 x5/16	9 to 12 in.	¾
2¼x ⅜	13 to 16 in.	¾
2½x7/16	17 in. and upward	¾ or ⅞

In general rivets should not be used in tension, that is, in stress that tends to pull the heads off. If it is necessary to use rivets in tension no less than four should be used in the joint, and these must be symmetrical with the application of the load. The angles used should be of thick metal, so that they will not bend under the load, preferably ½ in. or ⅝ in. thick.

For tension on rivet heads use no more than one half of the single shear value.

Separators are made either of short pieces of gas pipe or of castings. (See Godfrey's Tables, page 33.) These are the pieces that are placed between double beams to hold them a given distance apart and to take the bolts that united the beams. Usually separators in double beam work are placed about 4 or 5 feet apart. The office of separators in some cases is to distribute load that may be applied to one beam only of a pair, so that they will deflect together. In cases where all or nearly all of the load is delivered to one beam of a pair, as when floor-beams connect to the web of one beam of the pair, ordinary cast separators are not sufficient. In such cases there should be riveted diaphragms between the beams. These may be opposite the beam connections.

Beams resting on walls should have anchors at the ends. The usual anchor is a plain ¾-in. round rod 6 in. long for beams up to 10 in. and 12 in. long for larger beams. A hole is punched in the web of the beam 2 in to 4 in. from the end to receive the anchor. The anchor rod is usually kinked at the middle. A pair of 6x4 angles 2 or 3 in. long, riveted to the web of the beam, may also be used as an anchor.

Details in Timber Trusses. The details in timber work are very often neglected or given little consideration, or they may be left to the workmen to work out on the job.

The strength of bolts and spikes can not be so definitely determined as that of rivets in steel work. Some standdard, however, should be used. The following table is recommended for ordinary conditions in sound wood of the hardness of yellow pine. For white pine deduct 20 per cent.

TABLE II.

VALUE OF BOLTS OR SPIKES IN SHEAR.

Diameter in ins.—	1/8	3/16	1/4	5/16	3/8	1/2	5/8	3/4	7/8	1
Load in lbs.—	40	80	150	200	300	500	800	1200	1600	2000

Table. II is based primarily on the value of a spike or bolt in bending, for in the ordinary case the spike will bend in the wood before it will shear off. In using the term shear in the heading of the table it is meant to convey the idea that the stress on the bolt or spike is at right angles to the axis. It is assumed that the thickness of the wood will be such as to give proper bearing against the same, as, for example, not less than one-inch boards for 1/2-in. bolts, and not less than 2-in. boards for one-inch bolts. If the pressure is tranverse with the grain of the wood, use one-half of the values in Table I.

The distance between bolts along the grain and from a bolt to the end of a piece should not be less than about six times the diameter of the bolt.

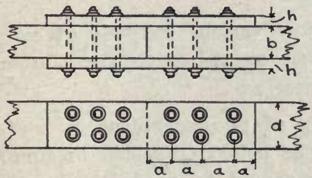


Fig. 6.

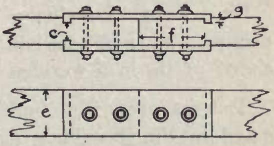


Fig. 7.

Figs. 6 and 7 illustrate two kinds of splices in wood. For full efficiency in the splice of Fig. 6 the sum of the widths of the two splicing pieces (if of wood) should be equal to the piece spliced, or $2h$ should equal b. However, the full tensile strength of members in wood is not often demanded. In a 4″x8″ piece with one-inch bolts the net section would be $4 \times 6 = 24$ sq. in. At 1,600 lbs. per sq.

in. this would take a tension of 38,400 lbs. The six bolts in double shear are good for $12 \times 2{,}000 = 24{,}000$ lbs. The distances a should be six inches.

The splice shown in Fig. 7 is with steel or cast iron plates having gibs at the ends. Here the bolts are used to hold the plates together. $c \times e$ measures the net area in tension. $2g \times e$ measures the area in bearing against the gibs, which has a value of 800 and 1,000 lbs. per sq. in. for white pine and yellow pine respectively. $2f \times e$ measures the area in shear along the grain. Wood is particularly weak in this respect, so that a comparatively large area is needed here. For white pine use 80 lbs. per sq. in., and for yellow pine use 100 lbs. per sq. in. for this shearing value.

One of the most important and difficult details to take care of in wood is this one, where the wood is in longitudinal shear. In many details the wood is notched, as for the inclined end post of a truss, and a tension is applied at this notch. Frequently the distance from this notch to the end of the piece is not sufficient to develop the tension of the piece at a proper safe shear on the fibers of the wood.

Figs. 8 to 15 inclusive show various methods of connecting the inclined end post or rafter to the bottom chord or tie in a wooden truss.

Trusses are often built up of two-inch plank as indicated in Fig. 8. They may be bolted or spiked together. Filling or separating blocks should be used at intermediate points in long compression members. There are several advantages in this kind of construction. Pieces can be more easily handled, details can be more readily made, and the lighter pieces are in better condition for seasoning.

The diagram in Fig. 11 indicates the method of finding the tension in the bolt. The side ba of the triangle is the stress in the rafter. On the same scale bc is the tension in the bolt.

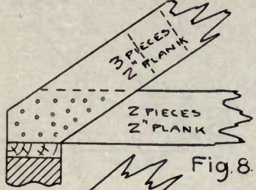


Fig. 8.

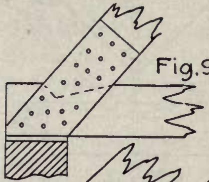


Fig. 9.

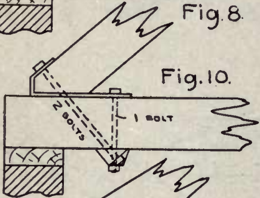


Fig. 10.

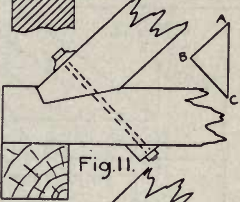


Fig. 11.

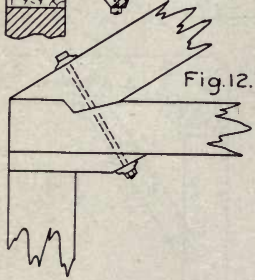


Fig. 12.

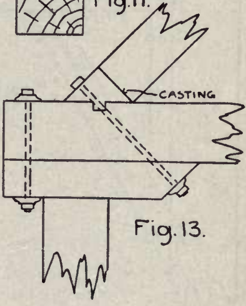


Fig. 13.

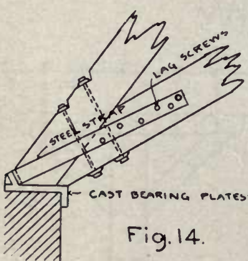


Fig. 14.

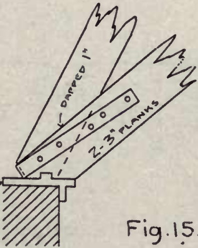


Fig. 15.

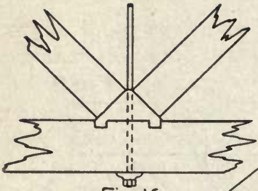


Fig. 16.

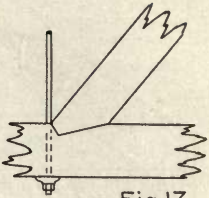


Fig. 17.

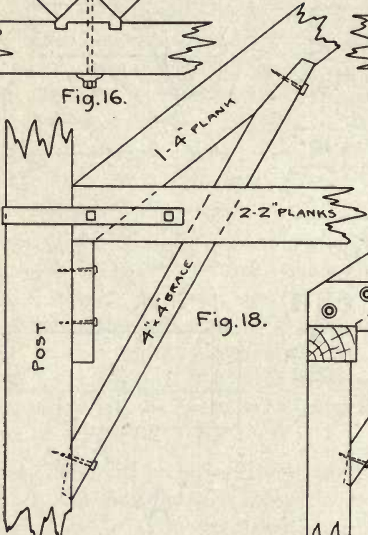


Fig. 18.

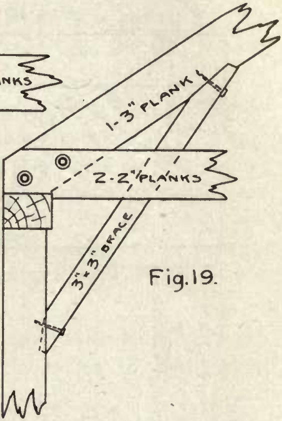


Fig. 19.

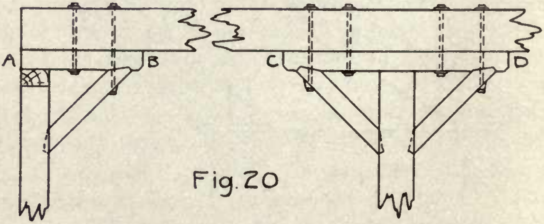


Fig. 20

Figs. 16 to 20 inclusive show other details in wooden construction.

Attention is called to the caps or corbels in Fig. 20. The one marked CD, together with the knee braces, could be counted upon to relieve the load in the timber beam above the post, if that load is a symmetrical one; but AB cannot offer such aid except by putting a bending moment in the post. It is an error to rely upon such construction as that shown to the left of Fig. 20 for any other purpose than to brace the building.

CHAPTER XI.

Estimating Loads.

For estimating the load carried by a beam or truss, use the following data:

Wood	4 lbs. per sq. ft. one inch thick
Stone concrete	13 " " " " " "
Cinder concrete	9 " " " " " "
Brick walls	10 " " " " " "
Stone walls (not granite)	13 " " " " " "
Granite	14 " " " " " "
Lime mortar	9 " " " " " "

Hollow brick arches weigh about 8 lbs. per sq. ft. per inch of thickness.

Ordinary tile arches weigh about 4 lbs. per sq. ft. per inch of thickness.

Tile partitions weigh as follows:

WEIGHT PER SQ. FT.

	2-in.	3-in.	4-in.	5-in.	6-in.
Semi-porous	12 lbs.	15 lbs.	16 lbs.	18 lbs.	24 lbs.
Porous	14 lbs.	17 lbs.	18 lbs.	20 lbs.	26 lbs.

Book tile or flat tile for ceilings and roofs are made in lengths of 16, 18 and 20 inches in 2-in. tile; 16, 18, 20 and 24 inches in 3-in, tile; and 24 inches in 4-in. tile. The 2-in. tile weigh 12 lbs. per sq. ft.; the 3-in. tile, 20 lbs. per sq. ft.; the 4-in. tile, 22 lbs. per sq. ft.

For wooden shingles on a roof allow 2¼ lbs. per sq. ft., for slate shingles allow 5 to 7 lbs. per sq. ft. For Spanish tiles allow 7½ to 8 lbs. per sq. ft. For tarred felt and gravel or slag allow 2 lbs. per sq. ft. for the felt and tar, 3 lbs. per sq. ft. for slag, and 4 lbs. per sq. ft. for gravel.

For slate tiles allow 14 lbs. per sq. ft. per inch of thickness. For solid clay tiles allow 11 lbs. per sq. ft. per inch of thickness

For corrugated steel in gages of 16, 18, 20 and 22, allow 3.6, 2.7, 1.9 and 1.5 lbs. per sq. ft. respectively.

In ordinary floor work the steel beams will weigh, in pounds per sq ft of floor, about one-third of the span in ft, and the girders one-fifth of their span in feet Thus, if the span of the beams is 15 ft., use 5 lbs. per sq. ft. for a trial weight of the beams; if the span of the girders is 20 ft., use 4 lbs. per sq. ft. for a trial weight of the girders.

For trusses carrying roof loads only use one-tenth of the span for a trial load per sq. ft.

For steel columns estimate the weight per lineal foot at about four times the area of the section in square inches.

Ordinary partitions in a building are usually considered as covered in the allowance for live load. When an allowance is made for their weight, it may be in a uniform load of say 5 or 10 lbs. per sq. ft. Fire walls around elevator shafts and the like are taken at their full weight for the beam on which they are built.

For exterior walls, estimate the weight per running foot for a solid wall and deduct the proportion of the wall occupied by windows or other openings.

The New York Building Code allows a reduction of the live load on columns carrying several floors as follows:

For top story use full live load.

For next story use full live load.

For each succeeding story deduct 5 per cent from full live load until 50 per cent of live load is reached. Use 50 per cent of live load for all remaining stories.

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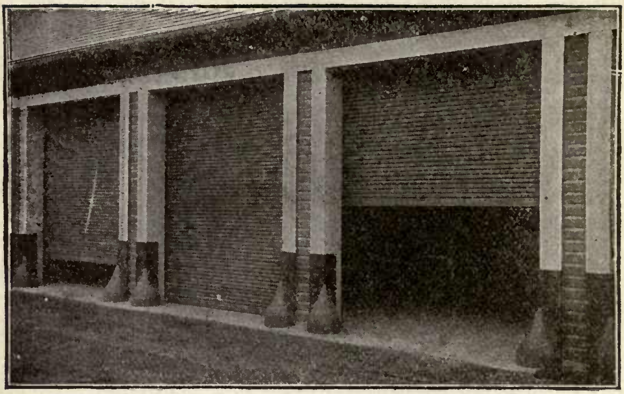

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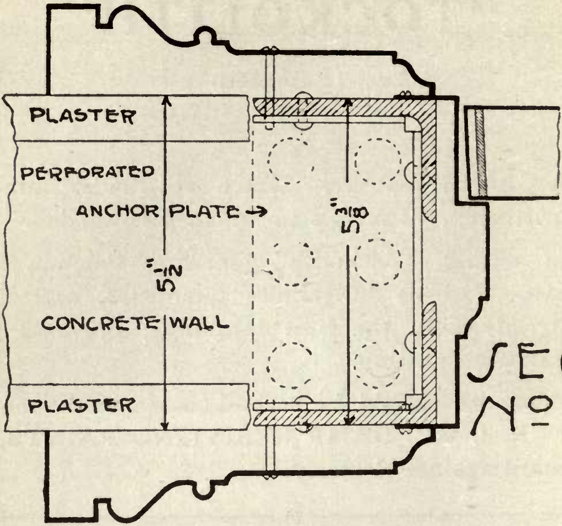
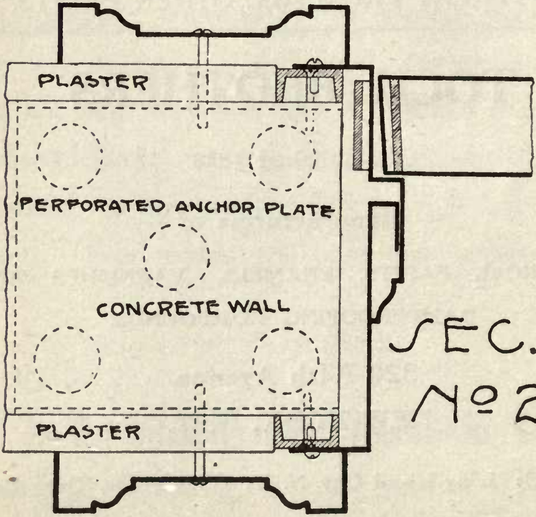


SEC. No 1



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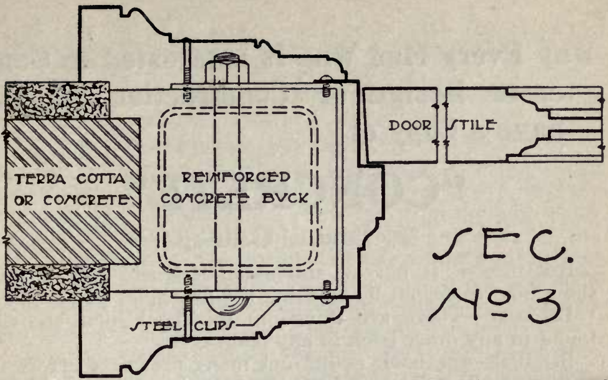

SEC. No 3

By the use of re-inforced concrete construction, the walls, floors and partitions of a building may be made fireproof. By the use of "Dahlstrom" Metal Doors and Trim the fireproofing is completed, and protection is provided for the contents and the lives of the occupants of such a building, making it "Absolutely" fireproof in reality, as well as in name.

The illustrations herewith show different methods of properly installing "Dahlstrom" Doors and Trim in reinforced concrete partitions.

In Section No. 1 is shown an angle iron frame connected by perforated anchor plates. The frames are shop made to proper size for the doors and are set in position before the concrete is poured. The concrete passing through the perforations will securely anchor the frame in position and affords a good fastening for the finished Metal Trim.

Section No. 2 shows perforated channel shaped clips of sizes to suit the thickness of the concrete. These are laid in the forms as the concrete is poured, leaving the 2″ flanges exposed on each side of the partition after the forms are taken down. A 1″x½″ channel is then applied by machine screws which are tapped into the flanges of the perforated clips. These channels will then serve as grounds for the plaster, and also to fasten the finished steel jambs to, which are made adjustable so as to take up any possible variation in the thickness of the partition.

Another method of construction for concrete partition is shown in section No. 3. In this case the bucks are of re-inforced concrete made up separately and erected before the partitions are placed. Provision for fastening the flanges of the steel jambs is made by sleeves or holes through the bucks, through which the fastening bolts can pass.

Additional information and submission drawings to meet special requirements will be cheerfully furnished by applying to

Dahlstrom Metallic Door Co.

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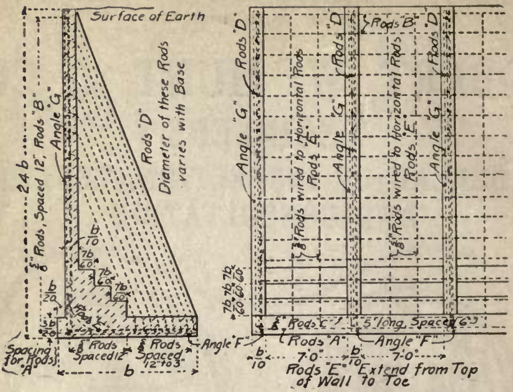

GODFREY PATENT RETAINING WALL

See Engineering News, Oct. 18, 1906.

See Engineering-Contracting, Dec. 21, 1910.

See Engineering-Contracting, Jan. 18, 1911.

A complete description of this retaining wall will be found in the above references, as well as the method of designing such walls. This is the safest design for reinforced concrete retaining walls that is before the public. It is also economical. For further information address Edward Godfrey, Monongahela Bank Building, Pittsburg, Penn'a.